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Accuracy in Mechanistic Pavement Design Consequent upon Unbound Material Testing

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Appendices

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Abstract

As part of a European Union funded research study (the "SCIENCE" project) performed between 1990 and 1993, granular road construction material and subgrade soil specimens were tested in the four participating laboratories of the project:

Laboratório Nacional de Engenharia Civil	Portugal
University of Nottingham	United Kingdom
Laboratoire Central des Ponts et Chaussées	France
Delft University of Technology	The Netherlands

The author was based the first of these and visited the other participating laboratories, performing the majority of the work described.

Inaccuracies in repeated load triaxial testing based on the use of different apparatus and instrumentation are identified. A detailed instrumentation comparison is undertaken, which results in the magnitude of potential errors being quantified.

The author has derived material parameters and model coefficients for the materials tested using a number of previously published material models. In order to establish these parameters a method for removing outliers from test data based on the difference between the modelled and experimental material parameters for each stress path applied was developed.

The consequences of repeatability and reproducibility, variability and inaccuracies in the output of repeated load triaxial testing, on the parameters and, hence, on computed pavement design thicknesses or life is investigated using a number of material models and the South African mechanistic pavement design method.

Overall, it is concluded that:

- Instrumentation differences are not as critical as variations in results obtained from different specimens tested in a single repeated load triaxial apparatus. It was found that specimen manufacture difference yielded greater variation that instrumentation differences.
- Variation in results has some effect on the upper granular layers, where higher stress levels are experienced, but even quite considerable variation in the results from materials used in the lower layers has little effect on pavement life.
- Analytical methods to determine the stresses and strains vary considerably as do the predicted pavement thicknesses consequent on using these methods.

The inaccuracies in testing (large discrepancies are found when the same material is tested in the same laboratory) and the limitations of the available material models severely limit the usefulness of advanced testing and non-linear modelling in routine pavement design. On the basis of this study it is recommended that a more simplistic pavement design approach be taken keeping in line with future developments of testing and modelling and field validation.

Acknowledgements

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Laboratório Nacional de Engenharia Civil	Portugal
University of Nottingham	United Kingdom
Laboratoire Central des Ponts et Chaussées	France
Delft University of Technology	The Netherlands

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ACCURACY IN MECHANISTIC PAVEMENT DESIGN CONSEQUENT UPON UNBOUND MATERIAL TESTING

1 AN INTRODUCTION TO THE ANALYSIS OF PAVEMENTS

1.1 HISTORY OF ROADS

Roads have been constructed almost since the invention of the wheel; 1800 years ago the Romans constructed a vast network over much of Europe {Croney and Croney (1991)}. As wheeled transport replaced pack animals more roads were constructed. Various construction methods were used from stone set, brick pavements, and wooden block pavements to the asphalt and concrete that the road pavement structures comprise today.

The engineers responsible for setting out these early roads would have known something of the elements of soil mechanics. They would have understood that it was necessary to remove poor strength material and replace it with superior material; this imported material required a loading capacity suitable for the proposed loads. Today's pavement engineering follows exactly this principle {Transport Research Laboratory (1993)} using the following three steps:

i) Estimate the amount of traffic loading that will use the road over the selected design life in years;

ii) Assess the load carrying capacity of the subgrade soil over which the road is to be built;

iii) Select the most economical combination of road pavement materials and layer thickness that will provide satisfactory service over the design life of the pavement without exceeding the subgrade load carrying capacity. It is usually necessary to assume that an appropriate level of maintenance is also carried out.

The road infrastructure has over the years, particularly in the last century, become one of Europe's most important economic assets. It provides door-to-door transportation for both people and goods. Recent rapid growth in road traffic numbers and gross weights of commercial vehicles may lead to premature failures of trunk roads and motorways, which were not designed for these loads {Loach (1987)}. Vast amounts of money are invested in the construction and maintenance of a country's road network emphasising the importance of good pavement design and management procedures. Repairs to roads are expensive not only because of the cost of repair but also because of the extensive delays to private and commercial road users. Poorly designed road pavements may cause premature failure, however over-designed pavements waste both limited funds and precious materials. There is a need to design roads for greater and greater traffic volumes while conserving the limited natural material resources and therefore a need for a better understanding of the behaviour of various materials that make up a road pavement structure.

1.2 DESCRIPTION OF THE ROAD STRUCTURE

Road pavement structures are built for the purpose of operating wheeled vehicles safely and economically, thus forming a reliable road transport system. Pavements comprise one or more layers of imported material placed over the existing soil. There are essentially three types of road pavement:

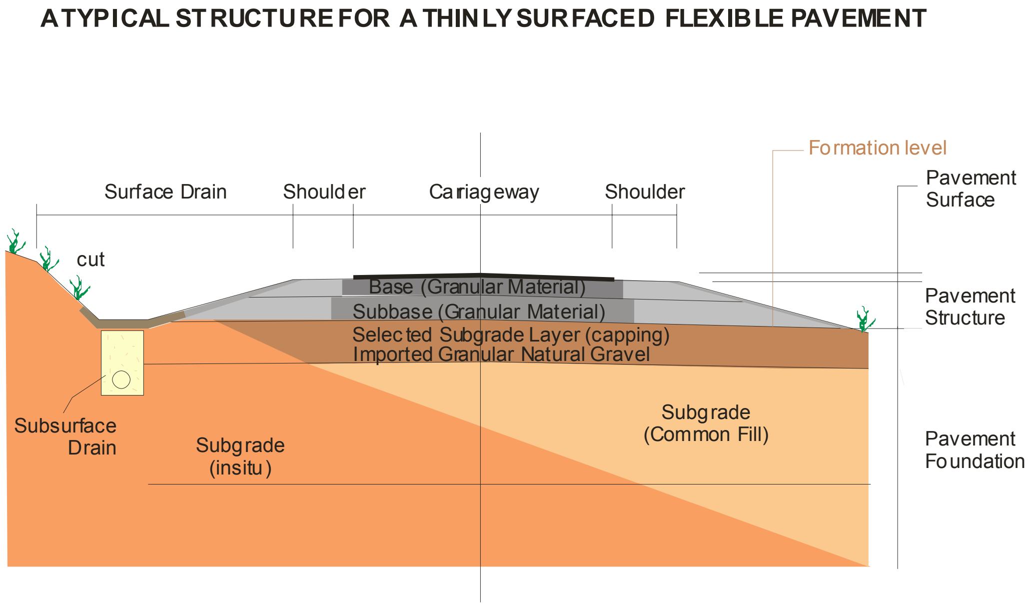
- Unsurfaced pavements with natural gravel wearing coarse surface;
- Flexible pavements these may have thick or thin bituminous surfaces, and;
- Rigid pavements these have concrete bases and surfaces.

This study will only consider flexible pavements since it is these that provide more than 90% of the road stock in most European countries. The structural format of a flexible pavement is shown in a schematic road cross-section in Figure 1-1.

These layers can be combined and simplified with the assumption that all road pavements have essentially three components, namely:

- The foundation;
- The pavement structural layers, and;
- The surfacing.

Figure 1-1 A Typical Pavement Structure for a Flexible Pavement



1.2.1 The Foundation

This component consists of the underlying subgrade soil (cut or fill) and a selected subgrade or capping layer. The pavement foundation is required to:

i) Carry construction traffic without significant rutting (i.e. a small number of cycles with larger stress applied).

ii) Be stiff enough to provide a firm platform on which to compact overlying layers.

iii) Be sufficiently stiff under normal traffic induced stresses (i.e. a large number of cycles with smaller stresses applied) and thus prevent excessive flexing of overlying layers which would thereby result in fatigue failure.

iv) Be resistant to permanent deformation within themselves under normal traffic induced stresses.

v) Not be frost susceptible, and in certain instances to provide adequate frost protection to susceptible *in-situ* subgrades.

1.2.2 The Pavement Structural Layers

The pavement structure comprises the major load bearing layers. These materials are almost certainly imported and often modified to achieve a required strength.

The subbase is the secondary load-spreading layer underlying the base. It will normally consist of a material of lower quality than that used in the base. This layer also serves as a separating layer preventing contamination of the base by the subgrade material and, under wet conditions, it has an important role to play in protecting the subgrade from damage by construction traffic.

The base is the main structural element of the road pavement. It is required to spread the wheel load so that underlying material is not over-stressed. The base in flexible construction may be of dense bituminous material or lean concrete but, in many cases for more lightly trafficked roads, cheaper Unbound Granular Material (UGM) may be adequate.

In summary these layers must, under normal traffic loading, be:

i) Sufficiently stiff to spread the load well and thus prevent over-stressing of underlying layers.

ii) Stiff enough to prevent fatigue failure of any overlying layers.

iii) Able to resist permanent deformation within their thickness.

iv) If bound, have adequate fatigue life.

A substantial amount of research has been done on bound base layers, both bituminous and lean concrete such that the structural behaviour of bound materials is relatively well understood and documented. Acceptable material models can be used readily in the various design methods available for example the Nottingham Pavement Design Method {Brown and Brunton (1990)}.

1.2.3 The Surface

The surfacing is principally to provide adequate protection to the base, to reduce water ingress into the pavement and to provide skid resistance and riding quality to vehicles that travel along the road. The surface generally has little structural significance and will not be considered in detail in this work. However, in the case where it will make some contribution to the structural integrity of the pavement, it will be taken to be a component part of the base layer. For example, when a 40 to 60 mm layer of dense bituminous mixture surface is used the base thickness will be effectively increased by an amount related to the thickness of the surface. The characteristics of the base will then be used for the total layer thickness.

1.2.4 Thesis Focus

This thesis concentrates on the unbound granular materials that are used for structural base and subbase layers in pavements and the unbound subgrade soils that occur in road foundations pavements (selected subgrade and subgrade). Details of the thesis scope and aims are given later in Chapter 1.6.

1.3 TECHNICAL BACKGROUND

In recent years a number of important empirical studies {Parsley and Robinson (1982); Paterson (1987); Chesher and Harrison (1987), Watanatada et al (1987)} have shown how the costs of operating vehicles depend on the surface condition of the road. The studies have also improved the knowledge of how the deterioration of roads depends on the nature of the traffic, the properties of the road construction materials, the environment, and the maintenance strategy adopted.

In present times, when economic considerations control road construction, every proposed road construction and rehabilitation project is economically analysed with rigour against many similar projects and only relatively few construction projects are funded. Ideally only the best quality materials would be used for the construction of road pavements, however these materials are expensive and the need to construct more economic roads is becoming more significant as material shortages occur. In the light of this it is very important to use the most economical materials possible for the construction, yielding a road that is neither over-designed nor will fail before the design period (the proposed period of usage) is complete.

1.3.1 Road Construction Materials

A wide variety of materials is used for the construction of road pavements. These vary from crushed quarried rock through to crushed and screened natural gravel. Good quality naturally occurring ‘as dug’ gravel can be mechanically stabilised and modified. Poorer quality naturally occurring materials are often either mechanically stabilised or stabilised using a chemical stabilising agent such as lime or cement (this involves the addition of a stabilising agent, mixing with sufficient water, compaction and final curing to ensure that the mechanical potential is realised).

In many areas good quality materials are rare and haul costs can be high, therefore more marginal quality materials may need to be used. In the case where these marginal materials do not comply with the common bearing capacity criteria, as set out in many design guides, designers often disregard these materials. These materials may be satisfactory for road construction, but a better understanding of their behaviour in a pavement structure is necessary in order to determine the limits of their applicability. It is under these circumstances that relevant laboratory testing, analytical modelling and full-scale field verification become essential.

An example of that approach can be found in many European countries where recycled materials are used for road construction. These materials vary from incinerated household refuse to demolished building rubble {Sweere (1990)} and specifications for the use of these materials have been drawn up based on extensive laboratory tests.

1.3.2 Pavement Design Methods

Knowledge concerning the characteristics of natural materials such as unbound granular layers and subgrade soils is still relatively limited. Many of the pavement design procedures presently employed remain empirically based. They were often developed from experience with existing roads, supplemented with the analysis of test sections and a few major research projects like the well-known AASHO Road Test {AASHO (1962), Powell et al (1984)}. Test methods for characterisation of the mechanical properties of the unbound pavement materials and the subgrade are often still empirically based and only yield a rough estimate of the fundamental material parameters required for pavement design. Consequently, material specifications too are mainly based on experience and practical considerations.

Choosing the correct road construction materials and having a full understanding of their material properties and performance under traffic loading is paramount for the successful utilisation of the road. This understanding is essential for the design of new roads as well as the addition of layers to roads requiring strengthening during rehabilitation works.

The empirical approach to material characterisation and pavement design has been used for many years; continual revision of these methods with newly gathered experience has improved on many early shortcomings. These empirical test and design methods form a sound basis for pavement design. Because of their empirical nature, they are often very well implemented and, most importantly, simple in nature. The testing techniques require only standard laboratory equipment and often the pavement design techniques use charts from which the pavement design for a given set of circumstances can be obtained.

In order to assess unbound materials for their suitability for road construction (crushed rock, natural gravel and subgrade soils), engineers and scientists have devised criteria to which materials must comply in order to qualify for a specific use in a road pavement structure. These criteria vary for different layers in a pavement. Generally the criteria are more onerous the nearer the surface they are intended to be used. The most familiar of the empirical, or experimental, methods to assess pavement materials used by road engineers is the Californian Bearing Ratio (CBR) test, which was developed in the USA, in the late thirties, for characterising the bearing capacity of soils and unbound granular materials {Porter (1938)}.

The major drawback of empirical methods is that they only operate within the limits of the experience on which they are based. Extrapolation from that experience to, for instance, higher axle loads or the use of marginal materials can lead to uncertainty in designs. It is thus desirable to develop more general analytical design procedures. These analytical methods should be based on the capability to calculate stress, strain or deflection in a pavement subjected to an external load providing pavement response that can subsequently be interpreted in terms of long-term pavement performance such as cracking and rutting {Sweere (1990)}.

1.3.3 Stress and Strain Determination

Determining stresses and strains in multi-layered pavement structures using the analytical solutions developed earlier by Boussinesq (1885) and Burmister (1943) became possible with the advent of computers in the 1980's. Programs like ELSYM5 {Federal Highway Administration (1985)} and BISAR {Shell Laboratorium (1972)} were developed and have allowed the calculation of stresses and strains at any point in a multi-layered pavement structure in response to an external load.

These methods, however, all use linear elastic theory to calculate stresses and strains and thus require a single value of Young's modulus and Poisson's ratio to be assigned to each layer. In reality both of these parameters vary throughout the pavement layers because the material properties are stress-dependent {Hicks and Monismith (1971); Brown (1979); Barksdale (1972a); Brown and Pappin (1981)}. Due to this stress-dependent non-linearity the conventional term Young's modulus is inappropriate and 'elastic stiffness' or 'resilient modulus' should be used. In general the term 'elastic stiffness' is used for bituminous materials and 'resilient modulus' for unbound granular materials and subgrade soils. Powell et al (1984) stated that the analysis of the behaviour of unbound granular bases in analytical terms presents considerable problems particularly for pavements where granular layers form the major structural element. They conclude that it is unacceptable to use constant values for elastic stiffness, resilient modulus and Poisson's ratio. Although, as a first approximation, dividing the granular base layer into a number of sub-layers and giving lower stiffness values to the deeper sub-layers allows the stress dependency to be partly simulated.

1.3.4 Shell Pavement Design Method

In Europe the Shell Pavement Design Manual {Shell International (1978)} is extensively used for pavement design. The pavement structure is divided into three horizontal layers, commonly called the asphalt, unbound granular base and subgrade, each with constant values of Young's elastic modulus and Poisson's ratio.

The Shell method states that the elastic modulus should be obtained from dynamic deflection measurements or from repeated load triaxial tests for the subgrade. Clearly there is a contradiction, since it is known that these materials have stress-dependent properties and a single value for these parameters is thus not possible. Fortunately, the Shell method gives an alternative, which is probably followed by almost all who use it, stating that when results of these sophisticated testing techniques are not available the stiffness parameter of the subgrade may be estimated from the CBR value. The stiffness of the granular base is taken to be a function of the thickness of the base layer and of the supporting layer. The stiffness of the asphalt layer is estimated from mix-properties of the asphalt, such as type of bitumen and void content. Poisson's ratio is simply chosen at a constant value for each of the layers. This is not unique to the Shell method and many analytical pavement methods provide 'typical' elastic properties for pavement materials. Such assumptions are likely to be inaccurate and, consequently, one can expect the results of the analysis to be inaccurate.

1.3.5 Finite Element Approach

The disadvantages of multi-layered linear elastic analysis can be overcome by using a finite element approach to the calculation of stresses and strains in pavements. A section of the pavement structure is divided both vertically and horizontally into a large number of small elements and an iterative process applied to each element which assigns stiffness parameters dependent on the stress level in the particular element. Of course material models must be developed to relate resilient modulus and Poisson's ratio to stress. Most commonly, the recent development of models uses data from repeated load triaxial tests since this laboratory test has proven to be the least complex within the research world. Although no generally accepted pavement design procedure based on repeated load triaxial tests is commonly available, some development of standard test methods for repeated load triaxial tests and subsequent pavement design are being compiled, as will be discussed in the next chapter.

The analytical determination of the response to external loads on a multi-layered pavement structure does not, in itself, constitute a mechanistic pavement design procedure. Long-term monitoring of in-service pavements is a requirement in order that the models are properly validated. Unfortunately this is a lengthy process since most pavements are designed to withstand many millions of load applications. Although accelerated testing might solve part of this problem, data on actual performance under normal traffic and climate are still required for a complete validation.

The primary benefits that could accrue from the successful application of mechanistic procedures {Yoder and Witczak (1975)} are:

i) Improved reliability for design.

ii) Ability to predict specific types of distress.

iii) The ability to extrapolate from limited field and laboratory results.

The ability to design a pavement for site-specific conditions will influence the amount of conservatism included in the design and more reliable design methods will result in optimum use of available funds.

The reliable prediction of pavement distress (e.g. cracking and rutting) in order to minimise the costs of maintenance and rehabilitation, is a major benefit of mechanistic design procedures.

The ability to extrapolate from limited amounts of field or laboratory data before attempting full-scale long-term projects would eliminate concepts that are thus determined as having very little merit.

The development of reliable analytical or mechanistic pavement design procedures also offers the following benefits over traditional empirical design methods:

i) The consequences of different loading conditions can be evaluated, thus the damaging effects of increased loads such as high tyre pressures and multiple axles can be modelled.

ii) The consequence of utilising available materials can be estimated, thus marginal or non-traditional materials can be simulated, and their future performance predicted.

iii) Diagnostic techniques can be developed which will allow better understanding of premature distress or conversely why some pavements exceed their design expectations.

iv) Improved diagnostic techniques will allow time related pavement effects to be included in designs, for example the effect of asphalt ageing and seasonal effects such as thaw weakening may be included in estimates of performance. Methods can be developed to better evaluate the long-term benefits of providing improved drainage to the pavement and road in general.

It is outside the scope of this dissertation to investigate all of these effects, but it is clear that a sound analytical pavement design using robust material parameters will provide designers with more confidence to design efficient, reliable and economic pavements which take into account all exogenous effects.

Because the behaviour of asphalt is better understood, this dissertation aims at providing more insight into testing and the determination of the resilient modulus of both granular materials and subgrade soils for use in analytical pavement design and to highlight any possible errors involved in the testing and subsequent analysis of the effect on the final pavement design.

1.3.6 Pavement Failure Mechanisms

The traffic carrying capacity of a flexible pavement is governed by how effective the pavement layers are in preventing:

- Fatigue cracking of the asphalt surfacing;
- Shear failure of the granular base and subbase materials, and;
- Wheelpath rutting resulting from subgrade failure.

This work primarily investigates the resilient behaviour of unbound granular materials and subgrade soils and the consequence of the resilient parameters on the life of pavements.

1.4 MATERIALS TESTED DURING THE STUDY

Numerous materials typical of those found in Europe and used in pavement construction were considered during this work. A complete list of these materials and their classification details, where known, are shown in Appendix A.

Exactly which specimens of each material were tested by the Author is shown in Table 1-1. This table presents which materials were tested and at which of the participating Laboratories the tests were conducted. The tests programmes were divided into five phases and the table also shows under which of these phases the tests were conducted. During the course of this work 101 specimens were tested at all of the participating laboratories and the Author personally manufactured and tested 56 of these. Further, he was present during a further 6 tests in the course of his travels between the laboratories.

Table 1-1 Materials Tested under the Various Test Programmes

Laboratory	Phase 1 Test Programme I					Phase 2 Test Programme II					Phase 5 Test Programme III					Phase 3 & 4 Artificial Specimen				
Material	LNEC	UNOT	LRCF	LRSB	DUT	LNEC	UNOT	LRCF	LRSB	DUT	LNEC	UNOT	LRCF	LRSB	DUT	LNEC	UNOT	LRCF	LRSB	DUT
London Clay (LOC)	✓ (red)	✓	✓			✓ (red)	✓ (red)	✓		✓	✓ (red)									
Seine et Marne Silt (LIM)											✓ (red)									
Fountainbleau Sand (SFB)		✓	✓	✓																
Hard Limestone (CCD)	✓ (red)	✓		✓										✓ (red)						
Soft Limestone (CCT)		✓		✓										✓ (red)						
Microgranite (MIG)				✓		✓ (red)	✓ (red)		✓ (red)	✓										
Artificial Specimen (PTFE)																✓ (red)	✓	✓	✓ (red)	✓

Note: Tests shown by red ticks were undertaken by the Author.

1.5 THE EUROPEAN 'SCIENCE PROJECT'

The majority of this work is based on work conducted during the 'Science Project' or more correctly 'A European Approach to Road Pavement Design' {Gomes Correia

(1996)}. A brief description of the 'Science Project' is contained in Appendix B. The technique of combining work from different laboratories and research institutions around the world is a good one as the individual workload and costs are less. Further, such co-operation highlights the problems of comparing work conducted with different apparatus, using different test methods and with different operators.

The main aims of the Science Project, initiated in 1989, were to co-ordinate and harmonise some of the activities of the different European material testing laboratories working in the field of pavement construction as follows:

i) To study the behaviour of subgrade soils and unbound granular materials during repeated load triaxial testing.

ii) To compare the performance of the different repeated triaxial apparatus.

iii) To compare the results of the different design methods using results obtained from the repeated load triaxial apparatus.

The four participating laboratories and their respective countries (and suitable abbreviations for each) as used in this dissertation are:

• Laboratório Nacional de Engenharia Civil	Portugal	LNEC
• University of Nottingham	United Kingdom	UNOT
• Laboratoire Central des Ponts et Chaussées	France	
Laboratoire Régional des Ponts et Chaussées	Saint Brieuc	LRSB
Laboratoire Régional des Ponts et Chaussées	Clermont Ferrand	LRCF
• Delft University of Technology	The Netherlands	DUT

1.6 SCOPE OF THIS DISSERTATION

This study focuses on the laboratory testing of a limited number of materials considered typical for road construction (unbound granular materials and subgrade soils) at different laboratories in Europe.

The reliability of an analytical pavement design method depends, at least, on data collected to define parameters within an established model. Such material properties need to be determined by laboratory means and, for this work, repeated loading triaxial testing is used since it closely simulates the loading and water content conditions expected in the road structure, as will be discussed in the next chapter. The suitability and limitations of this approach requires investigation, as does the reliability of the constitutive mechanical material model on which the subsequent pavement design depends.

The main objectives of this work are, therefore, repeated here:

1. To isolate and identify the various inaccuracies that may be introduced in repeated load triaxial testing of unbound pavement materials (unbound aggregates and subgrade soils) and, thus.
2. To determine the differences in the results obtained from different apparatus using the same materials under, as near as possible, identical testing regimes, thus making recommendations for improvements in the equipment and test procedures.
3. To determine differences in material parameters resulting from constitutive models for pavement materials (unbound aggregates and subgrade soils) analysis, thus recommending a sound method for analysis using repeated load triaxial test data.
4. To identify and quantify the effect of the material and model inaccuracies on pavement design.

In total, there were five ‘test phases’ conducted during the course of the ‘Science Project’, as follows:

• Phase 1	Test Programme I, the first inter-laboratory comparison was conducted at each of the five laboratories on a single specimen of each of three subgrade soils and three unbound granular materials	The Author conducted all of the testing of UGM and soils at LNEC
• Phase 2	Test Programme II, the second inter-laboratory comparison was conducted at each of the five laboratories on three specimens derived from a single sample of subgrade soil and unbound granular material	The Author conducted the UGM testing at LNEC, the UGM testing at LRSB and undertook visits to UNOT and DUT during their testing programme
• Phase 3	'Round-Robin' test procedure conducted on a single artificial specimen in most apparatus at each of the five laboratories	The Author conducted the testing at LRSB
• Phase 4	Instrumentation comparison conducted on the artificial specimen at LRSB using a number of different instrumentation methods	The Author conducted all of the testing at LRSB
• Phase 5	Test Programme III, comprised a test procedure conducted on two subgrade soils and two unbound granular materials at LNEC and LRSB respectively	The Author conducted the tests on soils at LNEC and UGM at LRSB

1.7 LIMITATION OF THIS WORK

Since work was conducted simultaneously at the various laboratories not all of the laboratory specimen tests were conducted by the Author (see Table 1-1). However, all of the test results conducted by all participating laboratories are analysed by the Author and are used in the subsequent comparisons and analytical design methods.

Not all of the possible materials that can be used for road construction are included here. It has been stated that this work concentrates on unbound materials (unbound granular materials and subgrade soils) for use in both pavement structural layers and pavement foundations.

Neither the material models nor the pavement design methods that use these models to predict stress and strains in the modelled pavement structure are analysed against full-scale performance. The analytical methods used here are used as tools to make comparisons between one material and another.

1.8 THE ORGANISATION OF THIS DISSERTATION

This dissertation is divided into ten chapters as follows:

This chapter is an introduction to the importance of roads and the need to conduct economical sound designs of the pavement structures of roads.

The following chapter, Chapter 2, describes flexible pavement design procedures (empirical and analytical or mechanistic) presently used in practice and research. Some discussion of the shortcomings of present design methods and the advantages of more sophisticated analytical methods is made. An introduction of the required laboratory tests to characterise road construction materials for use in these sophisticated design procedures is presented. Characteristic stresses are introduced as are the distribution of stresses in pavement structures. Lastly the quantification of flexible pavement structures under traffic loading in terms of life (axle loads) is made with respect to the behaviour of bituminous materials (surfaces and bases) unbound granular bases and subbases, and subgrade soils and selected layers or capping layers.

In order to acquaint the reader with the terminology used in this dissertation a review of the literature is conducted in Chapter 3 and 4. Previous work conducted on repeated load triaxial testing of pavement construction materials and modelling of the results is investigated. The main purpose of this literature review is to provide a detailed background of the technical field dealt with in this study. Chapter 3 concentrates more on the behaviour of road pavements under loading, discussing the stresses and strains in flexible pavements and the general behaviour of pavement materials under traffic loading. An introduction to the different repeated load triaxial apparatus configurations is made. Chapter 4 concentrates on the factors that influence the behaviour of materials in pavements. The mechanical behaviour of pavement materials can be greatly affected by water and the theoretical background of this influence is described. The basic method in assessing a pavement structure using a mechanistic approach is described. Also important is the compaction (density) of pavement layers and this is discussed. The effect of stress levels, material properties, load duration and frequency and loading history are also presented here.

Chapter 5 contains an introduction to the analytical, or mathematical, models that are used to predict the response of the road construction materials under traffic loading. Some further discussion of the stress dependency of these materials is made. A brief critique of the advantages and disadvantages of these models is made, which concludes with the reasons certain models were used in this work.

A detailed description of the repeated load triaxial apparatus that were used by each of the different laboratories is contained in Chapter 6. A comparison of these apparatus is made and their advantages and limitations are discussed. Measurement of the loads and associated deformations was conducted using most of the apparatus (smaller specimen size) by testing an artificial specimen, thus the relative accuracy of the various apparatus is discussed. Further, an experiment was conducted whereby different instrumentation was placed on a single artificial specimen and loads applied, again, based on the deformations measured by the instrumentation, certain conclusions are made. Some discussion about the statistical methods used herein to quantify the inaccuracies is made.

The materials investigated in this study are described in Chapter 7. The test procedures for each of the three test programmes are presented. The repeated load triaxial test results are presented and discussed. A description of comparative testing of an artificial specimen is discussed and conclusions drawn for this work. Conclusions and recommendations for an improved test method are made based on predicted errors for this type of laboratory testing technique.

In Chapter 8, the modelling of material parameters for pavement design is dealt with. Simple and complex models for predicting the behaviour of road construction materials are used to determine the various coefficients for use in the next chapter – pavement design. The differences in the analysis results are quantified and discussed.

Chapter 9 deals with the design of pavement structures using the parameters from the testing and subsequent analysis. The effects of the differences in the various test results from Chapter 8 in the final design of a typical pavement are discussed.

Chapter 10, finally, summarises the overall conclusions of this study and presents recommendations that focus on implementation of the knowledge presented in this dissertation.

2 FLEXIBLE PAVEMENT DESIGN PROCEDURES

2.1 INTRODUCTION

The improvement of analytical road pavement design methods requires a rational study of the mechanical behaviour of the constituent materials. It was stated in the previous chapter that the construction and maintenance of pavements is often uneconomic owing to inadequate understanding of the mechanical properties of pavement materials and foundation soils. This may lead to either uneconomic over-design, or under-design and hence to the early failure of the pavement. Improved knowledge about unbound granular materials and soil subgrades could provide a marked reduction in the construction and maintenance cost of flexible pavements.

Fundamentally there are three different methods of pavement design each requiring different material parameters:

- Empirical methods that use empirically determined material characteristics such as resistance to impact degradation and laboratory determined bearing capacity.
- Empirical methods that attempt to define an elastic stiffness or resilient modulus and Poisson's ratio for particular road construction materials based on some empirical determination and which analyse the pavement as a multi-layered structure. Often these elastic parameters are selected from tables determined from empirical data.
- True mechanistic, or analytical, pavement design methods that consider the stress dependency of the materials in each layer of a pavement structure by using sophisticated material testing and complex modelling. These methods are not commonly used in practice.

In the first method the occurrence of design inaccuracies would seem likely due to the material characteristic not accurately predicting the material behaviour under traffic loading. The second method improves on this but lacks consideration of the stress dependency of the materials and has a dubious link between the determination of elastic parameters and empirical laboratory tests. The third method is analytically sound but depends on the accurate determination of material parameters (and accurate criteria) to permit the models to predict the behaviour of the pavement under traffic loading accurately.

During any investigation for a road project there is inevitably some materials testing. These tests may be *in-situ*, conducted on the natural soil on which the road is to be constructed, for example probe testing. Alternatively, and more commonly, material tests may be conducted in the laboratory. Materials collected and transported to the laboratory would typically be borrow material for fills, naturally occurring gravel for pavement foundations and high quality materials such as crushed rock for construction of upper pavement layers.

2.2 EMPIRICAL PAVEMENT DESIGN METHODS

Many countries today still rely on empirical pavement design methods, realising that more sophisticated mechanistic design procedures often require complicated material testing techniques. It is recognised that unless these complex tests are performed too many assumptions regarding the material behaviour under traffic loading need to be made resulting in low confidence in the analysis and therefore little practical use. The best-known empirical pavement design procedure is that of the CBR method as discussed in the last chapter. More recently, the Transport Research Laboratory of the United Kingdom has published its Laboratory Report LR 1132 {Powell et al (1984)} which contains a design procedure for flexible pavements and the AASHTO method published by the American Association of State Highway and Transportation Officials {AASHTO (1993b)} which introduces the use of the resilient modulus to characterise subgrade soil support. However, the method states that if no equipment for the resilient modulus test is available the resilient modulus can be estimated from the results of simple laboratory tests. The South African pavement design method published by the Committee of State Road Authorities {CSRA (1983)} bases its charts on detailed mechanistic analysis.

2.3 MECHANISTIC PAVEMENT DESIGN METHODS

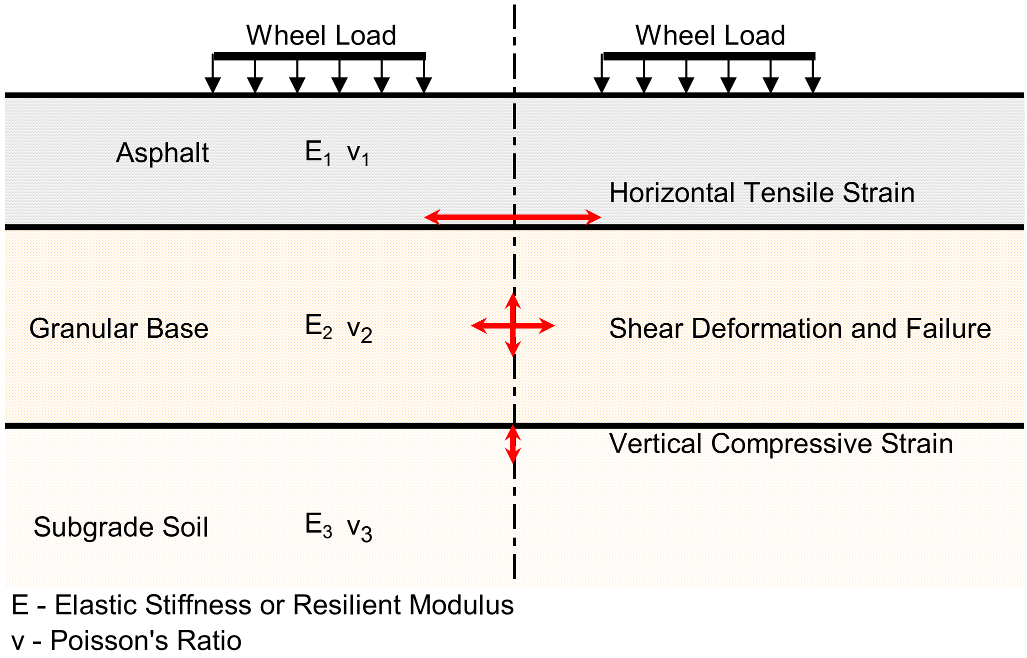
The mechanistic design or analysis of flexible pavements refers to the numerical calculation of the deflection, stress and strain in a multi-layered pavement when subjected to external loads and the subsequent translation of these analytical calculations of pavement response to the performance of the pavement. Performance of the pavement relates to physical distress such as cracking, rutting and roughness predictions.

Mechanistic design procedures are based on the assumption that a pavement can be modelled as a multi-layered elastic or visco-elastic structure, resting on an elastic or visco-elastic foundation. Assuming that pavements can be modelled in this manner, it is possible to calculate the deflection, stress and strain due to traffic loading at any point within or below the pavement structure. However, it must be recognised that pavement performance will be influenced by a number of factors that will not be precisely modelled by mechanistic methods such as rainfall, temperature, material quality and topography. It is, therefore, necessary to calibrate models with observations of performance and this is called empirical validation.

The number of flexible pavement design methods that make direct use of mechanistic design procedures is increasing and becoming more commonly used by engineers, for example, the Shell International {Shell International (1978)}, the South African Mechanistic Design Method [SA-MDM] {Maree and Freeme (1981)}, Kentucky Department of Transportation {Southgate, et al (1981)} and the Asphalt Institute Method {Asphalt Institute (1981)}. These new design methods all use the resilient modulus of the component materials in predicting pavement response under traffic loading. Further, all of these design methods have developed procedures for the general application to a variety of design considerations. Most of these design methods are primarily concerned with the two critical strain values namely the horizontal tensile strain at the bottom of the asphalt layer (to limit asphalt fatigue cracking) and the vertical compressive strain at the top of the subgrade (to prevent excessive permanent deformation). The SA-MDM method also considers shear deformation and failure in the unbound granular layers. These critical parameters are shown schematically in Figure 2-1.

It should be noted that although some design methods do not seem to consider a certain failure mechanism (such as failure due to shear deformation in the base) this omission may be satisfactory because the other failure mechanisms (such as those discussed above) may consistently be more critical. The particular properties of a poor layer will still influence failure in other layers since the analysis is conducted as a multi-layer structure. Indeed, it has been shown {Walker et al (1977)} that the life of flexible pavements is generally determined by cracking, which is caused by failure in the asphalt surface layer, or by rutting, which is related to the strain in the subgrade.

Figure 2-1 Pavement Failure Criteria for Mechanistic Design



A pavement design method based on an analytical principle considers the strains in the pavement structure caused by a standard axle load and these strains are subsequently compared to allowable strains. It is the determination of these allowable strains that require detailed long-term verification. Of course, it must be remembered that once validated for particular circumstances the parameters (allowable strains) will vary between locations due to climate, topography, material quality, etc. Sweere (1990) stated that with the exception of the SA-MDM few analytical design methods have been thoroughly validated with extensive field trials. The validation of the SA-MDM was conducted over some ten years using the Heavy Vehicle Simulator which loads actual pavements with a large rapid number of realistic wheel loads as described by Walker (1985).

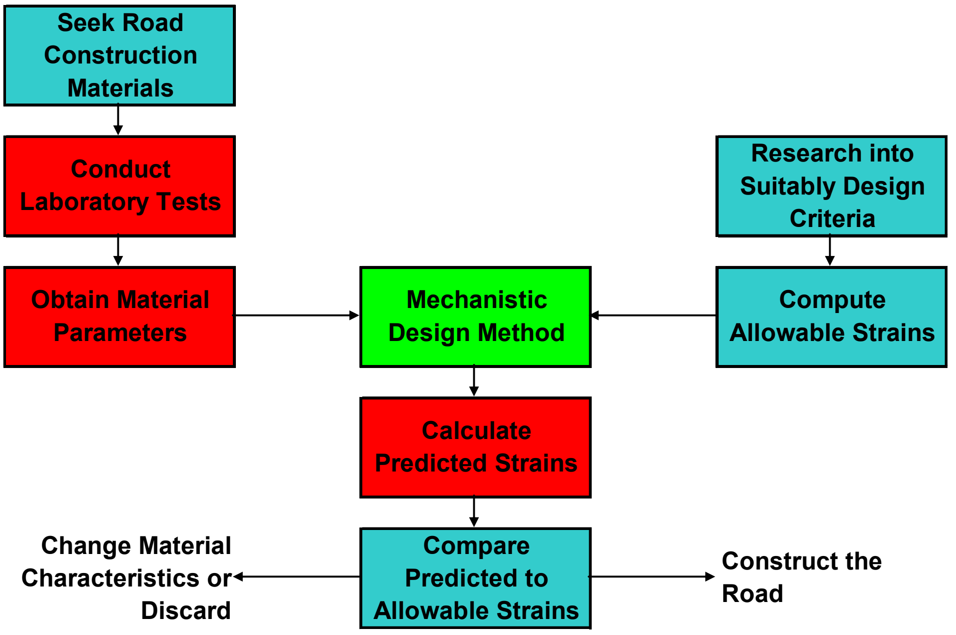
This work makes no attempt to verify the design criteria used by various pavement design methods. However, it is necessary to select a single design method and, once selected, this method is assumed to provide a satisfactory design standard. The purpose of this work is to obtain material parameters from laboratory testing that are suitable for use in the mechanistic design procedure and to quantify the potential errors in the final design due subsequent events, such as material sampling and testing.

Having reviewed the design methods, above, the SA-MDM methods was selected for use in this work, the main reasons for this are:

- This method considers the failure mechanism in the unbound granular base (or subbase) directly, although rarely directly critical, failure within this layer is often the cause of failure in another layer.
- Of the methods reviewed only this method has been thoroughly validated with extensive field trials.
- This method is ‘user friendly’ in that the equations of limiting parameters against vehicular loading are provided.

Therefore, as shown in Figure 2-2, this work undertakes to conduct the laboratory tests, obtain the material parameters and to calculate the predicted strains in order to determine the permissible loading for the particular pavement structure. It is clear therefore that some simple method by which the material parameters can be obtained is of prime importance in the mechanistic design procedure. The testing of relatively small samples in the laboratory is cheap and convenient but may also constitute a gross simplification of dubious accuracy. However, due to time and cost constraints engineers will inevitably favour these methods against more accurate field trails. The development of laboratory tests, simple or complex, endeavours to simulate conditions in the road pavement under traffic loading and to establish material parameters for verification against some criteria.

Figure 2-2 A Simplified Mechanistic Design Approach



Depending on the nature of the material used to construct a particular pavement layer there are certain recognised critical mechanisms of behaviour for which limiting values of stress or strain have been identified {Maree and Freeme (1981)}. A summary of these analytical structural criteria for multi-layer pavement behaviour for different construction material types is shown in Table 2-1.

It is desirable that pavement designs result in balanced structures whereby each layer contributes the pertinent mechanical properties rather than a single layer contributing much of the required performance with the other layers contributing little. Road pavement structures must be considered as entire structures, since commonly, failure will occur in a layer as a consequence of weaknesses in some other layer. For example a stiff cement stabilised base layer must be supported by a good foundation otherwise large stresses may form on the bottom of the rigid layer causing cracking and premature failure.

Table 2-1 Summary of the Analytical Structural Pavement Criteria

Layer	Material	Criteria	Location in the Layer	Mode of Distress
Surface	Bituminous	Horizontal tensile strain	Bottom	Fatigue cracking
Base	Cement or lime stabilised	Vertical stress Horizontal tensile strain	Top Bottom	Crushing Fatigue cracking
	Bituminous	Horizontal tensile strain	Bottom	Fatigue cracking and deformation
	Unbound granular	Principal stresses	Centre	Shear failure and densification
Subbase	Cement or lime stabilised	Vertical stress Horizontal tensile strain	Top Bottom	Crushing Fatigue cracking
	Selected	Vertical strain	Top	Deformation of the layer resulting in rutting
Subgrade	Selected	Vertical strain	Top	Deformation of the layer resulting in rutting

2.4 Laboratory Tests for Material Characterisation

Assuming that the pavement design criteria described above (Table 2-1) are correct and that it is possible to predict the occurrence of distress in a road pavement under some assumed traffic loading using these criteria, then it is likely that more accurate pavement designs will result. However this will depend critically on the accurate determination of the material parameters required by the analytical design methods or models to precisely characterise the material. It is likely that, for practical reasons, the determination of these parameters will be based on testing in laboratories.

2.4.1 Standard (Common) Laboratory Materials Tests

During unbound material investigations (bases and subgrades) for the use in road construction there exist relatively few common standard tests. Laboratory tests investigate the properties of compaction and bearing capacity, which attempt to simulate the material as compacted in the pavement structure. Common tests include the determination of the particle size distribution (grading) and the assessment of the clay content of materials, particularly soils, against a defined plasticity index. Some tests are used to characterise the degradation of unbound granular materials such as the Los Angeles abrasion test and the aggregate impact value test. All of these tests are of an empirical nature, developed to provide input data for empirical pavement design procedures or to provide a means of qualitative comparison of different materials. From the mechanistic pavement design aspect, compaction and bearing capacity have the most relevance since these values allow the designer to choose 'typical' elastic parameters for a specific material.

Compaction - Proctor Test

The 'Proctor test' {AASHTO 1993(a)} is widely used to determine moisture - density relationship of subgrade soils and unbound granular materials in the laboratory. This test was developed in the 1950s. Originally, the test involved compaction of soils in a 4 inch (≈102 mm) mould, using a drop hammer of weight 5.5 lbs (≈2.5 kg) dropped from a height of 12 in (≈305 mm) to apply the compaction effort. Since then a second level of compaction energy has been devised in order to simulate the better compaction equipment methods used. This method is called the 'modified Procter' compaction method and uses a drop hammer of 10 lbs (≈4.5 kg) dropped from a height of 18 in (≈457 mm) with a mould of 6 inches (≈152 mm).

Because of the impact of the hammer on the materials some degradation can occur, especially in the case of weaker aggregates. Due to the impact method of compaction there is some debate whether this test is a true reference value for field compaction.

Compaction - Vibrating Hammer Test

In order to simulate the methods of compaction applied during construction more realistically the Vibrating Hammer Test was developed {BS 5835}. This test uses a full-faced compaction plate in a 4 inch (≈102 mm) mould and a Kango-Hammer to supply the compaction energy.

Bearing Capacity - Californian Bearing Ratio

The Californian Bearing Ratio (CBR) is a very commonly known parameter for characterising the bearing capacity of soils and unbound granular materials {AASHTO 1993a}. This test was developed initially in the 1930s for the evaluation of subgrade strength. The laboratory CBR test is used throughout the world as a means of characterising qualitatively the bearing capacity of subgrade soils and unbound granular materials.

A plunger with a circular cross-section of 3 square inches ($\approx$1935 mm²) is driven at a specified rate into the material specimen compacted into a steel mould with a 6 inch ($\approx$152 mm) diameter. The forces required to penetrate to a depth of 0.1 and 0.2 inch (2.54 and 5.08 mm) are then expressed as percentages of the standard forces of 3000 and 4500 lb ($\approx$13.5 and 20.3 kN), respectively. To simulate the *in-situ* moisture in the laboratory, the CBR test can be carried out 'unsoaked' (at the compaction moisture content) or 'soaked' (after compaction the specimen is immersed in water for four days before testing).

The CBR value remains a common input value to pavement design procedures, such as the AASHTO (1993b) and TRRL methods {Powell et al (1984)} and even analytical pavement design procedures such as the Shell method {Shell International (1978)} rely on the CBR test to obtain, through empirical correlations, the fundamental stress - strain parameters required as input to the calculation of stresses and strains in pavements. However, in general, the CBR test is only conducted on natural gravel where the material will contain a relatively large percentage of fine-grained material. Crushed rock base material is assumed to have a CBR value of over 100% and therefore is always taken to be of satisfactory bearing capacity.

For the bearing capacity characterisation of subgrade soils, the CBR test is a reasonable means for assessing material strength, although the value obtained is only a relative measure of strength. During this test the deformation of the specimen is predominantly due to shear deformation and therefore the test provides an indirect measure of shear strength.

Because of its long-time worldwide use, the CBR test is also being used to obtain material stiffness parameters for input to analytical design procedures. In the absence of more accurate data empirical correlations between CBR and resilient modulus have been developed as follows:

$$Mr = a \times CBR^{b} \qquad \text{Eqn.2-1}$$

Where:

Mr	Resilient modulus	
CBR	Californian bearing ratio	
a, b	Constants dependent on material type, and have values:	

a	b	Researchers
10.0	1.00	Heukelom and Klomp (1962)
17.6	0.64	Powell et al (1984)

However these analytical procedures require fundamental material properties, such as resilient modulus, for input. It has been stated that this correlation does not consider the complexities of material behaviour and therefore often leads to considerable errors {Brown et al (1987), Sweere (1990)}.

2.4.2 Non-Standard Laboratory Testing

Logically, the best available approach to accurate evaluation of material properties in the laboratory is to simulate the dynamic loading condition to which a material is subjected in the field as closely as possible. Repeated load diametral tests are used quite extensively to evaluate the resilient characteristics of asphalt concrete. Repeated load bending tests are used to evaluate fatigue resistance of stabilised materials. The repeated load triaxial test is becoming more commonly employed for the evaluation of the resilient properties of granular materials and cohesive subgrades. However, other repeated load and cyclic tests have also been employed in evaluating pavement materials such as torsion, simple shear and hollow cylinder tests.

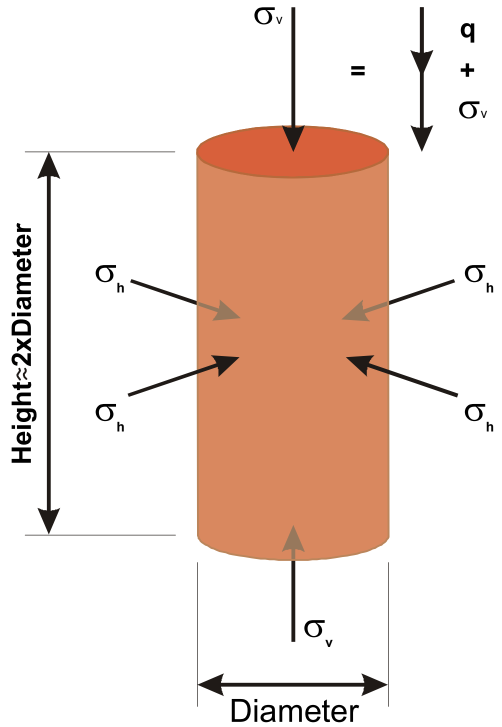
There is some reluctance by engineers and commercial laboratories to change from the relatively inexpensive simple characterisation tests described above to the complexity of non-standard material testing. Therefore the expensive and sophisticated apparatus (described below), which require a high level of operational expertise, are almost exclusively used for research purposes. However, due to its relative simplicity, the repeated load triaxial test has become the test most widely used for determination of resilient and permanent strain properties of road construction materials.

It is reported by Barksdale et al (1990) that in 1990 about 45% of transportation agencies in the United States of America used the resilient modulus testing for asphalt concrete, base, or subgrade materials.

Repeated Load Triaxial Tests

The general principle of a repeated load triaxial test is to subject a cylindrically shaped specimen of material to repeated compressive stress in both the axial and the radial directions, shown in Figure 2-3, thus simulating the traffic loading conditions on the pavement as closely as possible. The axial load simulates traffic loading while the all-round confining pressure simulates the lateral stress caused by the overburden pressure and the wheel loads.

Figure 2-3 Schematic Representation of a Triaxial Specimen under an Applied Load



Where	σ_v	Vertical stress;
	σ_h	Horizontal stress.
	q	Deviator stress

A major step towards implementation of the repeated load triaxial test has been the standardisation of the test by various organisations worldwide such as CEN (2000), Australia Standards (1995) and AASHTO (1994). These test methods give a description of equipment, specimen preparation procedures and testing procedures.

Only one of these three test methods requires the measurement of the radial strain (CEN) and thus do not provide all of the required information to accurately model the materials, although it may be argued that this is a considerable improvement on the relationships between CBR and resilient modulus discussed above. The triaxial test and the apparatus will be discussed in more detail in a forthcoming chapter.

Other Laboratory Tests for Determining the Elastic Parameters of Materials

The limitations of the triaxial test are that only two of the maximum of six stress components are varied independently for complete general conditions {Hyde (1974), Pappin (1979), Chan (1990)}. Only the vertical and horizontal stresses can be applied and this simulates the situation when the load is directly above the element, as will be discussed in detail in the following chapter.

Shear box and simple shear apparatus can be used in conjunction with the triaxial apparatus to increase the amount of information available on a particular material, and this goes some way to providing information with regard to the response to shear stress of the material {Pappin (1979)}.

The hollow cylinder apparatus allows a confining stress and an axial deviator stress to be applied in the same way as the triaxial apparatus. However, it is also possible to apply a torque and vary the pressure in the centre of the cylinder from that outside the cylinder {Thom (1988), Chan (1990)}. Application of a torque generates shear stresses on the horizontal and vertical planes in the wall of the cylinder, whereas variation of internal pressure imposes variation in circumferential stress. While this improves the reproduction of the pavement stress state, it makes for a much more complex and expensive test. Also, the test cannot be carried out on most pavement materials at their normal gradings since the wall thickness of the cylinder is necessarily rather small, namely 28 mm as described by Thom (1988) and Chan (1990).

All of the participants of the 'Science Project' were involved in repeated load triaxial testing in some form or another albeit using different materials, specifications and test methods. Consequently, only repeated load triaxial testing was conducted for this work and thus no further mention of the hollow cylinder, shear box or simple shear tests will be made.

2.4.3 Verification by Field Testing

Even though the mechanistic design procedure for a particular site may have been developed using basic material properties, there are still numerous assumptions and simplifications that must be made. It is obvious that an analytical pavement design could be conducted using approximations and recommended values. In reality, however, this would probably be very erroneous due to the particular conditions on site. It is fundamentally important that material testing is conducted in order that calibration of the predictive models utilised in the mechanistic design procedure be implemented. At present most mechanistic design procedures actually include a combination of mechanistic and empirical predictive models that are used in the design process, for example, climate factors and ageing predictions that must be considered empirically.

The use of full scale accelerated loading devices allows pavement deterioration to be observed in the field. An extensive programme was carried out in South Africa using the Heavy Vehicle Simulator (HVS) as reported by Walker (1985). Several units of this mobile device were used on sites in different locations to test sections of pavement in their 'as built' condition. By using high wheel loads repeatedly and continuously over several weeks, the equivalent of many years' of traffic loading was applied. A vast data bank was generated by the HVS test programme and this formed the basis for the South African pavement design system {NITRR (1985)}. Theoretical modelling was used to interpret the research results and extend them to design as is described by Maree and Freeme (1981) and Freeme et al (1982) and discussed later in this chapter. A similar accelerated loading device and test philosophy to the South African method is used in Australia {Metcalf et al (1985)}.

This work is only concerned with the testing of materials to determine material parameters that can be used for the consequent modelling of hypothetical pavement structures and therefore does not consider exogenous influences such as climate, etc.

2.5 THE QUANTIFICATION OF MATERIAL PARAMETERS FOR USE IN MECHANISTIC PAVEMENT DESIGN METHODS

When comparing the results obtained from laboratory tests conducted on material specimens, it is convenient to have some numeric parameter value (something that is met by CBR or plasticity index for example). It was stated that many pavement design methods assume that the construction materials have a linear elastically response to loading and thus their properties can be expressed in terms of constant stiffness and Poisson's ratio values. Engineers are familiar with values that characterise materials, providing a benchmark whereby they may classify materials and make decisions. Engineers easily appreciate that a material with a CBR=35% is subbase quality and would conclude that this material is probably a natural gravel with an acceptable clay content, they can easily characterise that it is a superior material to one with a CBR=5% which may be a clayey sand only suitable for common fill.

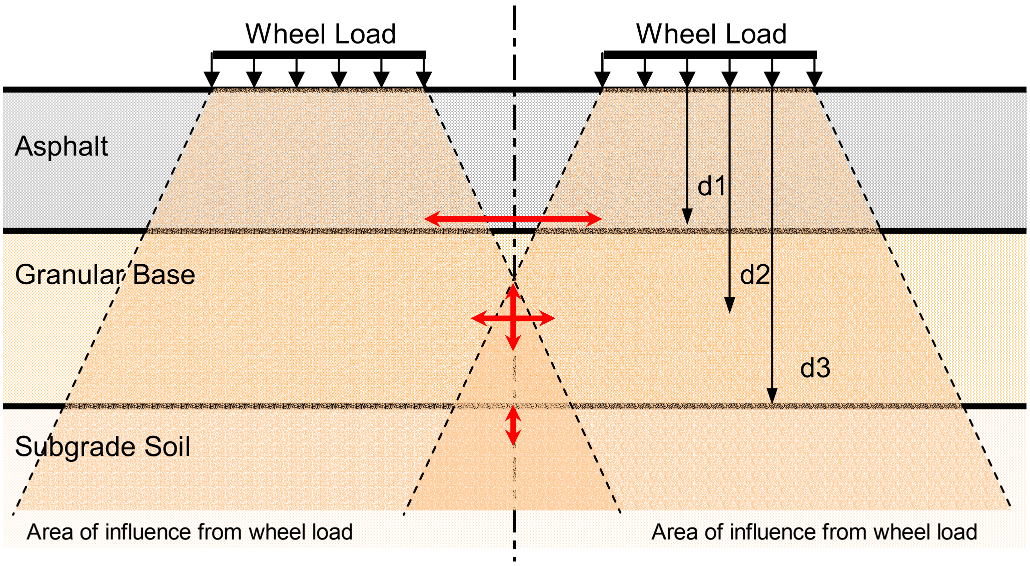
However, since road construction materials are really stress dependent, if they are to be characterised with elastic parameters (resilient modulus and Poisson’s ratio) some particular stress condition should be defined in addition to the density and moisture content of the material after construction. This is a familiar concept for engineers, who work to specifications that often define some arbitrary condition under which materials are classified. Indeed the familiar CBR test is defined by the force required to penetrate a plunger to two arbitrary depths in a specimen expressed as percentages of two arbitrary forces. Therefore it is reasonable to define some level of stress for which the elastic parameters for a material can be determined. This level of stress is called a “characteristic stress” in this work.

In order to determine the resilient modulus and Poisson’s ratio (dependent variables) the stress levels (independent variables) must be defined. This is not a new concept in mechanistic pavement design since it is common to provide ‘typical’ values for resilient modulus and particularly to define a constant value for Poisson’s ratio for a specific material, and these would have to have been taken at some arbitrary stress level.

2.5.1 Characteristic Stresses

All pavement structures comprise a number of different material layers each with different properties. Considering a pavement structure similar to that shown in Figure 2-1 where the accepted critical points in a pavement structure are shown. Those three critical points are shown as red arrows in Figure 2-4. Clearly each block, or material element in the pavement, experiences a unique magnitude of stress from traffic loading. The traffic loading, which is applied from the surface, is greatest at the surface. Also each element encounters a unique magnitude of stress from the overburden pressure, the overburden pressure increases with depth. The variation in stress magnitude is not restricted to change in depth but also varies along the same horizontal plane.

Figure 2-4 Stress Levels Applied at Different Points in a Pavement



In this work the term 'characteristic' is used to indicate that a unique stress situation is to be considered representative for the set of circumstances under which it was being accessed. In other words, if an unbound granular material, of base or subbase quality, was to be characterised using a representative value for the resilient modulus then this value should be obtained under a characteristic stress regime. The characteristic stress regime is then taken to be what this material might experience in a ‘standard’ pavement under traffic loading.

One of the tasks in this work is to compare the results of similar laboratory testing on identical material samples; therefore it is necessary to have some numeric value that is the result of the test. A characteristic resilient modulus for two samples when determined using identical analytical models fulfils this need perfectly and this is used throughout this work. The characteristic stress levels that are applied to the various material types will be defined later in this chapter (Section 2.6.4).

2.5.2 Distribution of Stresses in Pavements

The loads and forces applied to a solid body (such as a soil mass) are distributed within the body as stresses. Within a single layer of material it is assumed that the stresses vary smoothly and continuously throughout the body. However there is a variation from one layer to another, which is dependent on the properties of the material making up the layer. To control the expected behaviour of the road pavement within predetermined performance criteria, it is necessary to have an understanding of how the construction material behaves in individual layers as well as how the road pavement functions as a mechanism (layered structure) and this will be discussed in the next chapters.

2.6 THE QUANTIFICATION OF FLEXIBLE PAVEMENT STRUCTURES UNDER TRAFFIC LOADING

Taking the simple pavement structure as above it was shown that there are fundamentally three criteria for which it is necessary to quantify the pavement and these are discussed in the following sections. Considering that a pavement structure is analysed as a structure, it must be appreciated that each layer will have an effect on the other layers. The method of quantifying a pavement structure in most analytical design methods is by the amount of traffic loading that the structure can withstand until failure. Of course, the critical value is the lesser of the three failure criteria.

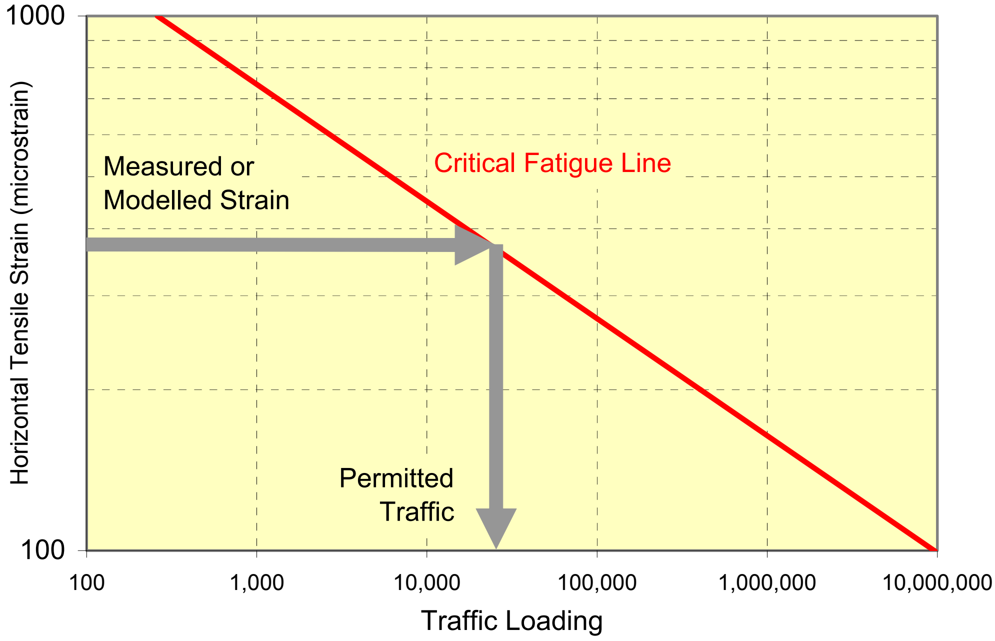
2.6.1 The Behaviour of Bituminous Surfaces and Bases

Bituminous materials are visco-elastic and under repeated traffic loading may either fail by cracking, fatigue or deformation or a combination of the above. The elastic stiffness of a bituminous mixture depends on the temperature, the rate of loading, layer thickness and the depth below the surface. The elastic stiffness of the material can be estimated with the aid of a nomograph as defined by Van der Poel (1954) or by laboratory testing, as described by Brown and Brunton (1990). Fatigue failure criteria, based on the limitation of the maximum horizontal tensile strain at the bottom of the bituminous layer at various material stiffness, air voids and temperature, have been developed from extensive laboratory testing. The Nottingham Pavement Design Method {Brown and Brunton (1990)} defines relationships for maximum allowable tensile strain for a given 'life' in terms of number of load applications. In the SA-MDM, Maree and Freeme (1981) also define fatigue failure criteria that are based on the limitation of the maximum horizontal tensile strain at the bottom of these layers at various material stiffness, air voids and temperature. These failure criteria have been laboratory determined and therefore appropriate shift factors must be applied to compensate for differences between laboratory and the field behaviour {Freeme et al (1982)}. Further, since traffic induced cracking is expected to begin at the bottom of the layer, some allowance has to be made for the traffic that can be carried before cracking becomes visible at the surface.

Figure 2-5 shows an example of the predicted relationship between the maximum horizontal tensile strain and the life of the pavement generally used by mechanistic pavement design methods.

In addition, ageing of the bitumen binder due to environmental influences will increase the stiffness of the binder, especially near the surface, which will result in the layer becoming more susceptible to cracking. It follows that the temperature under which the layer is expected to function is an important factor in the characterisation of the layer. Obviously, the properties of the asphalt material (percentage of binder, grading and density) also influence the stiffness of the material.

Figure 2-5 Trend for the Relationship between Horizontal Tensile Strain (Fatigue) Criteria and Traffic Loading for Asphalt Surfacing and Base



Maree and Freeme (1981) list approximate elastic stiffness values for asphalt at representative vehicle speeds and surface temperature, shown in Table 2-2. Table 2-3 {Freeme (1983)} contains a list of effective stiffness values for different pavement/ material states. Clearly, from these tables it can be seen that the stiffness decreases with increasing temperature and increases with increasing thickness and depth.

Table 2-2 Approximate Stiffness Values for Asphalt at Representative Vehicle Speeds and Surface Temperatures

Operating speed (km/h)	Depth from the surface (mm)	Asphalt Stiffness (MPa)			
		Gap-graded		Continuously graded	
		20°C	40°C	20°C	40°C
80 - 100	0 - 50	4,000	1,500	6,000	2,200
	50 - 150	6,000	3,500	8,000	5,000
	150 - 250	7,000	5,500	9,000	7,500
40 - 60	0 - 50	4,000	1,500	5,000	2,000
	50 - 150	4,500	3,000	6,000	4,000
	150 - 250	5,000	4,000	6,500	5,500

Table 2-3 Approximate Stiffness Values for Varying Asphalt Mixes

Material Grading	Depth from the surface (mm)	Asphalt Stiffness (MPa)					
		Good condition or new		Stiff dry mixture		Very cracked condition	
		20°C	40°C	20°C	40°C	20°C	40°C
Gap graded	0 - 50	4,000	1,500	5,000	1,800	1,000	500
	50 - 150	6,000	3,500	7,000	4,000	1,000	500
	150 - 250	7,000	5,500	8,000	6,000	1,000	500
Continuously graded	0 - 50	6,000	2,200	7,000	4,000	750	500
	50 - 150	8,000	5,500	9,000	6,000	1,000	750
	150 - 250	9,000	7,500	10,000	8,000	1,000	750

2.6.2 The Behaviour of Unbound Granular Bases and Subbases

Pavement layers constructed with unbound granular materials may be used in the base, subbase and even selected layers. The characteristics of these layers will depend on the quality of material used to construct the layer and the specifications to which the layers are constructed. Typically, crushed rock will be used for upper layers and natural gravel in lower layers. The characteristics of these layers depend on the quality of parent material used to construct the layer and the specifications (compaction) to which the layers were constructed.

Pavements that comprise base layers of untreated granular material often have a relatively thin bituminous surface layer (40 - 100 mm), and it is in these circumstances that the characteristics of the unbound granular material are critical to design. Most pavements have granular subbase layers and although it is important during construction to provide a working platform on which materials can be transported, laid and compacted, these layers are lower in the pavement structure and thus the stresses are reduced and their behaviour less critical.

Approximate values of the resilient moduli for granular material are given in Table 2-4 {Freeme (1983), updated by Jordaan (1993)}.

Table 2-4 Approximate Resilient Moduli for Granular Materials at Various Moisture Conditions

Material Description	Resilient Moduli (MPa)			
	Dry, well compacted, good support	Dry, well compacted, poor support	Wet, good support	Wet, poor support
High quality crushed stone	450 (250 - 1,000)	150 -600	50 - 250	40 - 200
Crushed stone	400 (200 - 800)	100 - 400	50 - 200	40 - 200
Crushed stone	350 (200 - 800)	100 - 350	50 - 150	40 - 200
Gravel base quality	300 (100 - 600)	75 - 350	50 - 150	30 - 200
Gravel	250 (50 - 400)	40 - 300	30 - 200	20 - 150
Gravel	225 (50 - 200)	30 - 200	20 - 150	20 - 150

Poisson's ratio is 0.35

Of course, there is some contention when particular values are published, such as those shown above, since the resilient moduli are stress dependent. These values do, however, give engineers some indication of likely magnitudes for values at likely stress values expected in pavements under traffic loading as discussed. The table also provides different ranges of values for different moisture conditions at different densities and levels of support.

Of the analytical design methods discussed earlier, only the SA-MDM considered that cumulative permanent deformation or inadequate stability in granular bases under repetitive traffic loading might be critical. In this method both of these distress modes have been related to the shear strength of the material {Maree and Freeme (1981)} and therefore, by limiting the allowable shear stress in the layer, distress can be avoided. The allowable shear stress under a single wheel load is calculated from the maximum single load shear strength, expressed in terms of the Mohr - Coulomb strength parameters, apparent cohesion and the angle of internal friction, and a selected ‘Factor of Safety’ as determined from laboratory triaxial tests. The general trend of the relationship with traffic loading is shown in Figure 2-6. This term, Factor of Safety, is somewhat misleading and a better term might be 'Granular Load Factor', however Factor of Safety will be used herein to avoid confusion with the SA-MDM. The Factor of Safety at any point in the layer has been defined {Maree (1978), Maree (1982)} such that:

$$\text{Allowable Shear Stress} = \frac{\text{Maximum Shear Strength}}{F} \qquad \text{Eqn.2-2}$$

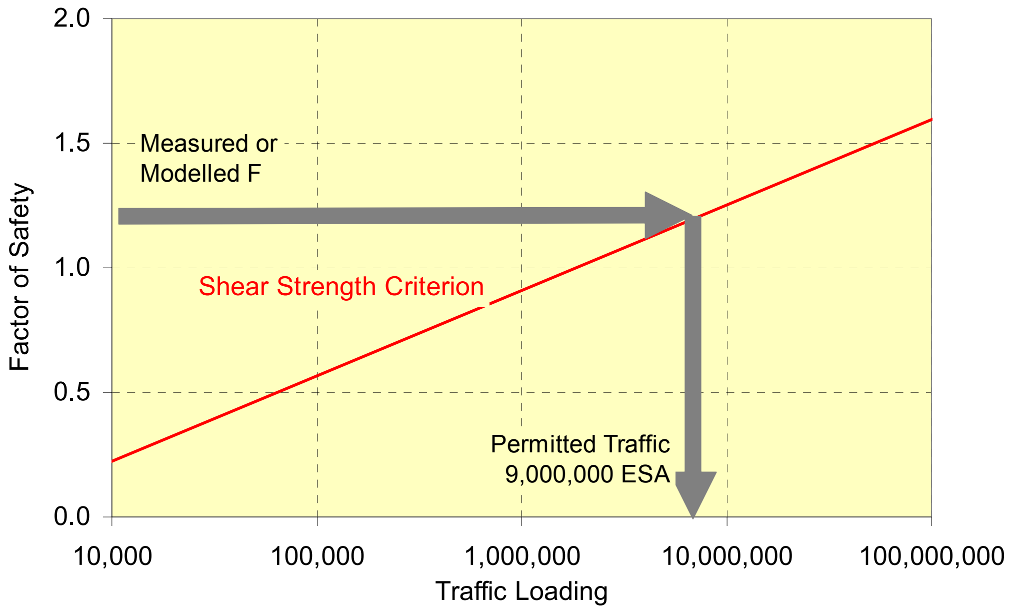
Where: F Factor of Safety

It was shown by Maree that:

$$F = \frac{\sigma_{3w}\left[K\left\{\tan^2\left(45+\frac{\theta}{2}\right)-1\right\}\right] + 2\times K\times c\left\{\tan\left(45+\frac{\theta}{2}\right)\right\}}{\left(\sigma_{1w}-\sigma_{3w}\right)} \qquad \text{Eqn.2-3}$$

Where:	c	Cohesion
	φ	Angle of internal friction
	K	Material constant (suggested values are 0.6 for highly saturated conditions and 0.95 for normal conditions)
	σ_{1w} & σ_{3w}	Calculated major and minor principal stresses acting at that point in the layer (allowable stresses) (with comprehensive stresses positive and tensile stresses negative)

Figure 2-6 Trend for the Relationship between Factor of Safety (Shear Strength) Criterion and Traffic Loading for Unbound Granular Materials



The SA-MDM suggests that the Factor of Safety be calculated at the mid-depth of a granular layer and at a point one sixth of the layer thickness from the bottom of the layer of the base layer under one of the wheel loads and at a vertical line between dual wheel loads.

2.6.3 The Behaviour of Subgrade Soils and Selected Layers

In addition to surface cracking, another principal measure of pavement behaviour is permanent deformation. Since the early 1960s the vertical elastic strain at the top of the subgrade soil, calculated with linear elastic theory, has been used to develop limiting strain criteria to control permanent deformation in this layer and therefore the rutting of the pavement {AASHO (1962); Walker et al (1977); Maree and Freeme (1981)}. Although there is a shortcoming with this approach (notably that the permanent plastic behaviour of pavement structures is related to resilient behaviour) the magnitude of stress in the subgrade at formation level is strongly influenced by the stiffness of the subgrade; moreover subgrade resilient modulus affects the level of stresses generated in all the pavement layers constructed on top of this layer. The resilient modulus of the subgrade is strongly influenced by moisture conditions.

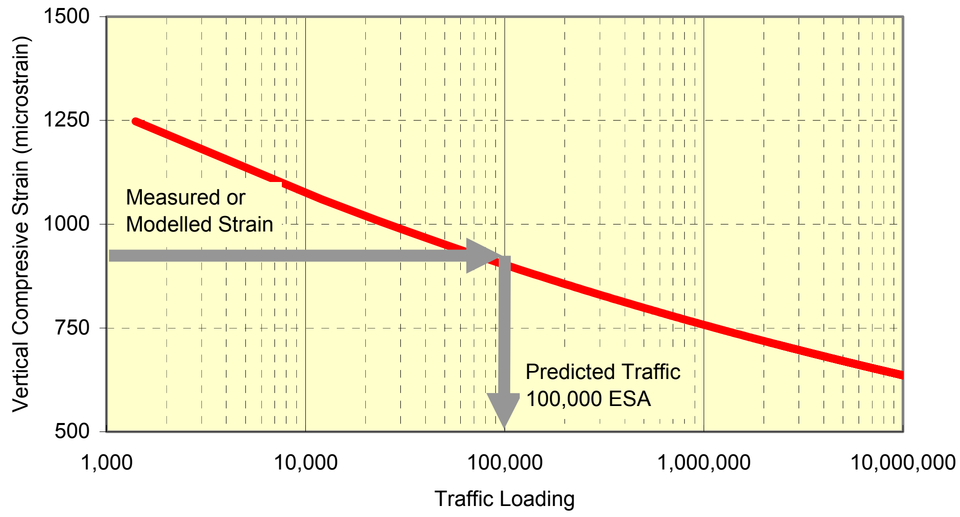
A table of typical resilient moduli of subgrade materials is given Table 2-5 {Jordaan (1993)}. Again there is some contention because fixed values are assigned to materials whereas these materials are stress dependent.

Table 2-5 Approximate Resilient Moduli of Subgrade Materials at Different Moisture Conditions

Material	Soaked CBR (%)	Resilient Modulus (MPa)	
		Wet state	Dry state
Gravel-soil	<15	20 - 120	30 - 200
Gravel-soil	<10	20 - 90	30 - 180
Gravel-soil	<7	20 - 70	30 - 140
Gravel-soil	<3	10 - 45	20 - 90

Figure 2-7 shows a typical design relationship linking resilient axial strain at the top of the subgrade with a permissible traffic loading level.

Figure 2-7 Trend for the Relationship between Compressive Strain Criteria for Subgrade Deformation and Traffic Loading

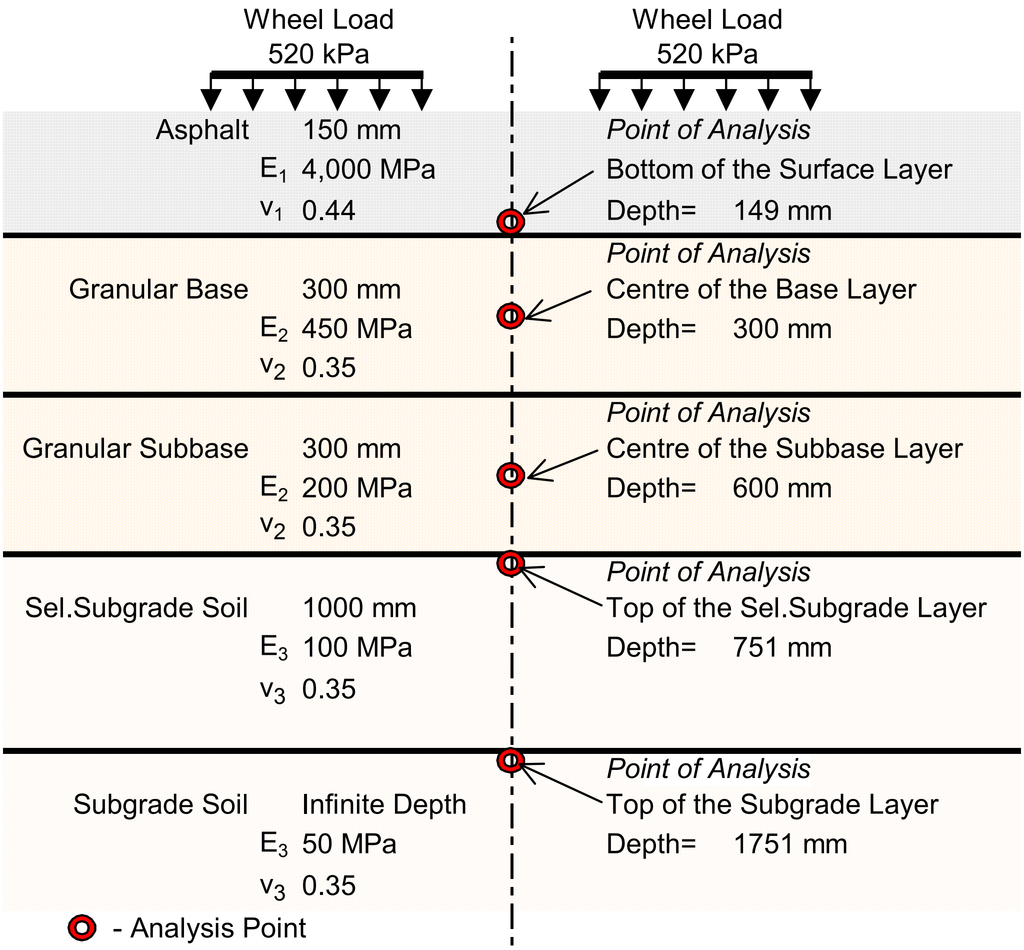


2.6.4 The Magnitude of Characteristic Stress

As was described in an earlier section, the levels of stress due to traffic loading reduce with depth in a pavement structure. In order that realistic laboratory simulation of the loading conditions is conducted in the laboratory it is necessary that characteristic stresses be applied to the particular material. Therefore it is reasonable to assume that greater loads would be applied to unbound granular materials used higher in the pavement structure and lesser loads for subgrade soils.

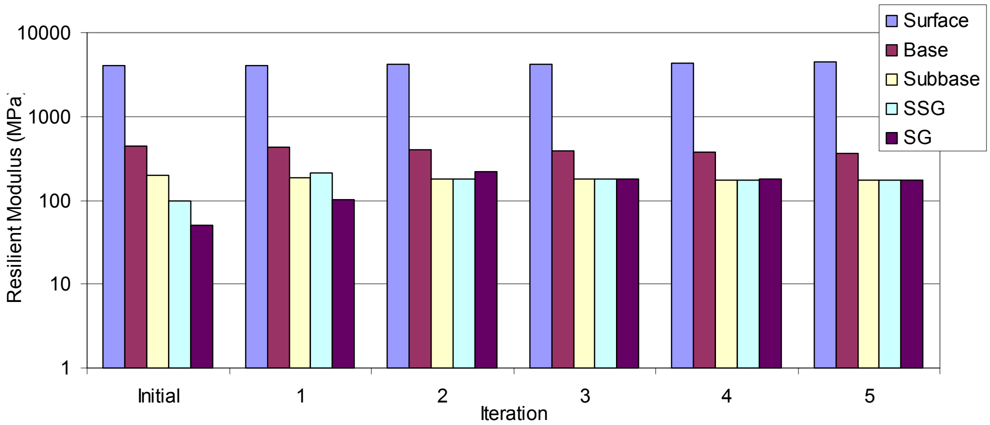
For this work, a stress level has been selected based on a ‘characteristic’ design pavement structure of specified layer thickness and elastic parameters (resilient modulus and Poisson’s ratio) for the materials that make up each layer. Taking the traffic loading as described in this chapter, the magnitude of the ‘characteristic’ stresses were calculated using ELSYM5, for a 20 kN dual tyre load with a circular contact area with a radius of 111 mm, and an assumed resilient modulus and Poisson’s ratio for the layers, based on recommended values from the previous tables. A typical European pavement structure was assumed and analysed as shown in Figure 2-8.

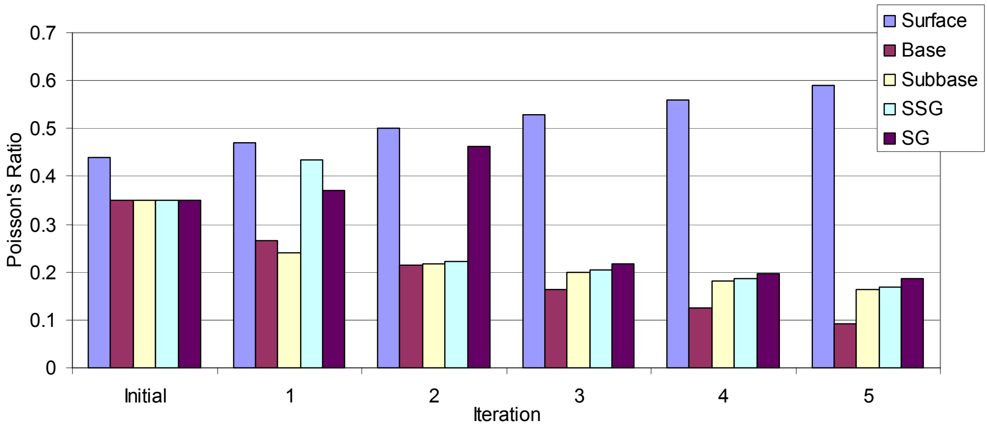
Figure 2-8 Materials and Pavement Details for the Calculation of Characteristic Stresses



It is recognised that the material parametric values (resilient modulus and Poisson's ratio) obtained from this first analysis should be re-entered and the pavement re-analysed using ELSYM5 with the new parameters and so on until an insignificant change in the parameters between this analysis and the previous analysis is obtained. This analysis was conducted and although the Resilient Modulus and Poisson's ratio varied somewhat, as shown graphically in Figure 2-9, the characteristic stresses in the base and the subgrade only varied by approximately 4 kPa between the initial iteration and the last. As a consequence of this, and the fact that the characteristic stresses are somewhat arbitrary anyway, the initial values were used for characterisation of materials and pavements in this work.

Figure 2-9 The Sensitivity of the Resilient Moduli and Poisson's Ration values to Re-Analysis





This analysis is shown in detail in Figure 2-10 for the self-weight calculations and Figure 2-11 when the pavement is loaded with a 20 kN dual tyre load. Taking the stress-path configuration (which will be described in detail in the next chapter) there exists a particular stress level (p_1;q_1) applied to a particular element in the pavement before vehicular loading due to self weight of the material. There is also a point on the stress path (p_2;q_2) that represents maximum loading due to the self weight plus the load from the vehicle passing over the element. It is these coordinates that are defined in these figures.

Figure 2-10 The Self Weight Characteristic Stress within a Typical European Pavement Structure

Self Weight Characteristics: **5-Layer Structure**

	Mean p1 =	Deviator q1 =
Bottom of the Asphalt:	2 kPa	2 kPa
Centre of the Granular Base:	5 kPa	3 kPa
Centre of the Granular Subbase:	9 kPa	7 kPa
Top of the Selected Subgrade:	11 kPa	8 kPa
Top of the Subgrade:	23 kPa	17 kPa

Compresive stress is positive

In the Pavement Structure Anisotropy - Ratio $\sigma_1:\sigma_3$ 2 : 1

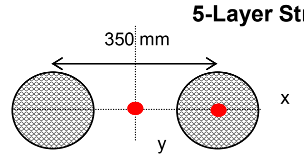
Layer	Thickness	Description	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Peak Deflection on the Surface			
1	150	Stress - Bottom of the Surface Layer, Depth= 149 mm	3	2	2
2	300	Stress - Centre of the Base Layer, Depth= 300 mm	7	3	3
3	300	Stress - Centre of the Subbase Layer, Depth= 600 mm	13	7	7
4	1000	Stress - Top of the Selected Subgrade Layer, Depth= 751 mm	16	8	8
5	Inf.	Stress - Top of the Subgrade Layer, Depth= 1751 mm	34	17	17

Based on this analysis characteristic stresses that are typically applied to an unbound granular base and subbase and those typically applied to a subgrade soil are calculated and are shown in red in Table 2-6. These stresses can be defined as the independent variables for which the elastic parameters (dependent variables) for the materials are calculated. This will allow a comparison to be made between materials and specimens since the same stresses will be applied.

Figure 2-11 The Characteristic Stress within a Typical European Pavement Structure

Loading Characteristics: **5-Layer Structure**

Dual Wheel Load =	20	kN
Tyre Contact Radius =	111	mm
Stress under each Wheel Load =	520	kPa



The Characteristic Stresses for a Typical European Pavement are:

	Mean Normal Stress p1 =	Mean Normal Stress p2 =	Deviator Stress q1 =	Deviator Stress q2 =
Bottom of the Asphalt:	2 kPa	-272 kPa	2 kPa	638 kPa
Centre of the Granular Base:	5 kPa	14 kPa	3 kPa	53 kPa
Centre of the Granular Subbase:	9 kPa	9 kPa	7 kPa	24 kPa
Top of the Selected Subgrade:	11 kPa	9 kPa	8 kPa	24 kPa
Top of the Subgrade:	23 kPa	22 kPa	17 kPa	21 kPa

Loaded (Under the Wheel) Compresive stress is positive

Layer	Thickness	Peak Deflection on the Surface	$\delta_{d(175,0)}$= 0.260 mm		
1	150	Stress - Bottom of the Surface Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 149 mm ν= 0.44 M_r= 4000 MPa	120	-427	-516
2	300	Stress - Centre of the Base Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 300 mm ν= 0.35 M_r= 450 MPa	42	-6	-8
3	300	Stress - Centre of the Subbase Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 600 mm ν= 0.35 M_r= 200 MPa	11	-5	-6
4	1000	Stress - Top of the Selected Subgrade Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 751 mm ν= 0.35 M_r= 100 MPa	8	-7	-7
5	Inf.	Stress - Top of the Subgrade Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 1751 mm ν= 0.35 M_r= 50 MPa	2	-2	-2

Loaded (Between the Wheels)

Layer	Thickness	Peak Deflection on the Surface	$\delta_{d(0,0)}$= 0.262 mm		
1	150	Stress - Bottom of the Surface Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 149 mm ν= 0.44 M_r= 4000 MPa	94	-130	-377
2	300	Stress - Centre of the Base Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 300 mm ν= 0.35 M_r= 450 MPa	43	-4	-8
3	300	Stress - Centre of the Subbase Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 600 mm ν= 0.35 M_r= 200 MPa	12	-5	-6
4	1000	Stress - Top of the Selected Subgrade Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 751 mm ν= 0.35 M_r= 100 MPa	8	-7	-8
5	Inf.	Stress - Top of the Subgrade Layer	σ_1(kPa)	σ_2(kPa)	σ_3(kPa)
		Depth= 1751 mm ν= 0.35 M_r= 50 MPa	2	-2	-2

Table 2-6 Determination of the Characteristic Stresses for a Characteristic Pavement

Pavement Layer (Material)	Thickness (mm)	Location within the layer	Characteristic Stress Invariants (kPa) p_1	p_2	q_1	q_2
Asphalt	150	Bottom	2	-272	2	638
Granular Base	*300*	*Centre*	*6*	*15*	*4*	*54*
Granular Subbase	300	Centre	12	12	9	26
Selected Subgrade	*1000*	*Top*	*14*	*12*	*10*	*26*
Subgrade	Infinite	Top	26	25	19	23

2.7 SUMMARY

The benefits of implementing mechanistic, as opposed to empirical, design procedures for new pavement construction, reconstruction, or rehabilitation are many. The key benefit is in providing the designer with powerful tools to evaluate the performance (specific distress types) of different pavement designs, instead of relying solely on limited empirical correlations or opinions.

A number of different pavement design factors such as material quality, moisture conditions and layer thickness can be examined using an analytical or mechanistic design approach giving these methods the potential to improve pavement design and to provide more reliability to designs.

It is recognised that laboratory tests that seek to simulate conditions in the road pavement are gross simplifications. Some correlation is possible from empirical laboratory test results to the elastic parameters required for analytical pavement design, however these correlations are inadequate. Laboratory tests have been developed using more sophisticated apparatus such as repeated load triaxial, hollow cylinder, shear box or simple shear tests, however there is some reluctance by engineers to change from the relatively inexpensive simple characterisation tests described above to the complexity of non-standard material testing. These complex test procedures are almost exclusively used for research purposes although the repeated load triaxial test is becoming the test most widely used for determination of resilient and permanent strain properties of road construction materials and international standards are being compiled for its commercial use.

The performance testing of full-scale pavements, ideally under controlled conditions of wheel load, moisture, climate and temperature, play an essential part in the development and calibration of the analytical pavement design. Pavement designs that are based on the parameters obtained from laboratory testing and the consequent analytical design methods require practical verification over a long term in the field.

When comparing the results obtained from laboratory tests conducted on material specimens it is convenient to have some numeric parameter value. In order to use the resilient modulus or Poisson's ratio some arbitrary stress conditions must be defined. These are taken as representative stress levels as would be experienced in a 'typical' pavement structure. These stresses are defines as 'characteristic stresses', for which the characteristic elastic parameters for a particular material can be determined.

The number of flexible pavement design methods that make direct use of mechanistic design procedures is increasing. All of these design methods have developed procedures for the general application to a variety of design considerations, however most of these design methods are primarily concerned with the two critical strains values namely the:

- Horizontal tensile strain at the bottom of the asphalt layer (to limit asphalt fatigue cracking), and;
- Vertical compressive strain at the top of the subgrade (to prevent excessive permanent deformation).

Of the design methods investigated the only method that presented a means of assessing the shear deformation and failure in the unbound granular layers for pavements with thin asphalt surfaces (40 – 60 mm) constructed on granular bases was the SA-MDM, which is therefore used here.

Of course, once validated for particular circumstances the parameters (allowable strains) and consequent designs will vary between locations due to climate, topography, material quality, etc.

3 STRESSES AND STRAINS IN ROAD PAVEMENT MATERIALS

3.1 INTRODUCTION

Pavement materials, such as unbound granular aggregate layers and soil subgrades, that make up the layers of a pavement structure exhibit two distinct types of behaviour when placed under the traffic loads:

- Elastic behaviour, which determines the load spreading ability of the layer, which manifests as cracking due to fatigue of the upper layers, and;
- Permanent deformation, which causes a build up of irrecoverable deformation, which after a number of load applications becomes apparent as rutting.

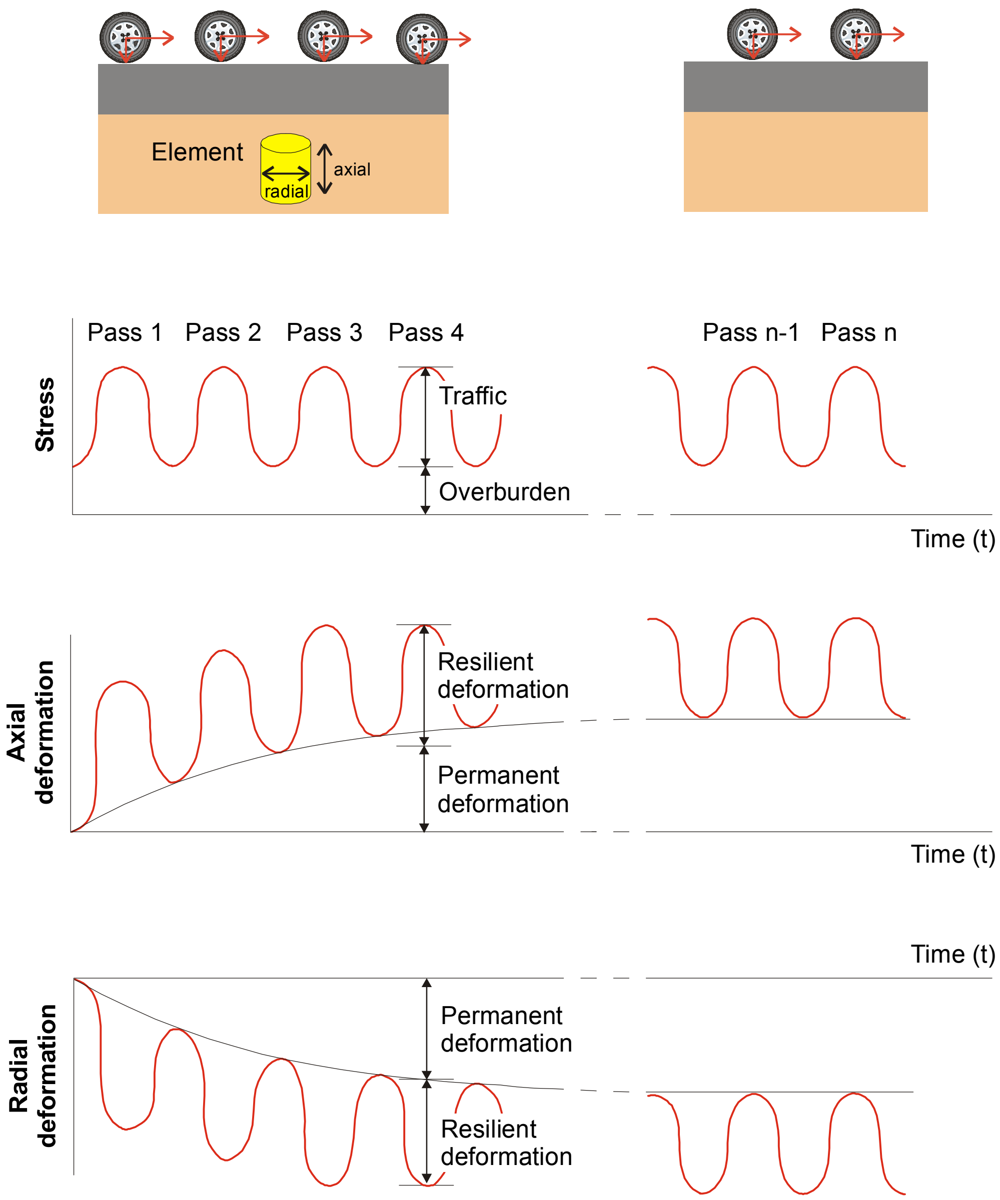
Consider a cylindrical element in a pavement as shown in Figure 3-1. As a vehicle passes over the cylindrical element a stress pulse is applied to it. These stress pulses are applied repeatedly in large numbers for the duration of the life of the pavement. For simplicity, it is assumed that each successive load is of equal magnitude. However, because this is not the case, an equivalency technique is used to reduce the actual or likely spectrum of applied loads to an ‘equivalent’ number of loads of a fixed magnitude (See Section 3.2.2).

The cylindrical element will also experience a constant stress due to the overburden pressure of the material above it.

The element will deform in both the axial and radial direction with each stress pulse. As shown in Figure 3-1 the elastic deformation recovers after each load. There is, however, there is a small permanent deformation applied to the element at each load cycle.

At an increasing number of load repetitions, the deformation of the material becomes almost entirely recoverable. The diminishing accumulating non-recoverable deformation from each of the stress applications is the permanent deformation {Thom (1988)}.

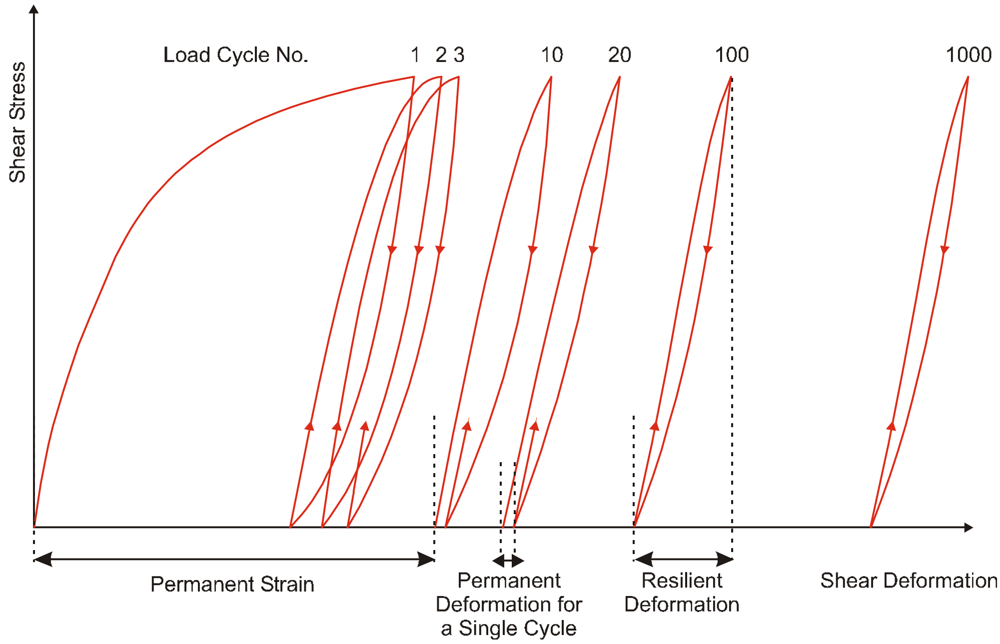
Figure 3-1 Loading in Pavements under Traffic



These deformations, or strains, are illustrated by means of a stress strain curve as shown in Figure 3-2. This figure shows the hysteresis loop behaviour experienced by the element during each load cycle. It also shows how, for a single cycle, the resilient deformation remains almost constant while the permanent deformation reduces with the number of load cycles {Thom (1988)}.

Based on this figure, the behaviour of the material can be seen to depend on the loading characteristics, i.e. number of loads and the magnitude of the load. In order to characterise these materials accurately the stress dependency should be considered.

Figure 3-2 The Stress-Strain Behaviour of Materials under Repeated Loading.



3.2 STRESSES AND STRAINS IN FLEXIBLE PAVEMENTS

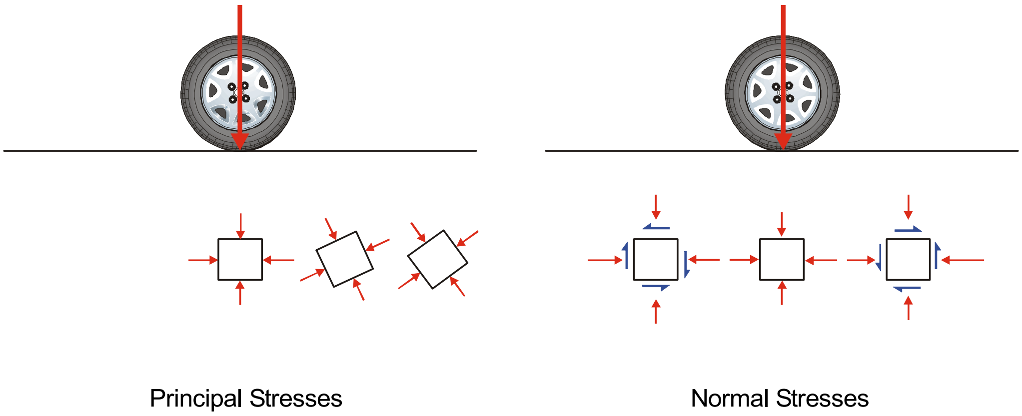
The structural capacity of a road pavement, i.e. the traffic loading that it can carry before its surface condition reaches one or more of pre-defined terminal levels, as discussed in the last chapter, is determined by the most critical structural behaviour in one or more of the material layers which make up the pavement. In general, road pavement distress manifests as either cracking or rutting.

3.2.1 General Three Dimensional States of Stress

Considering a cubical element within a pavement structure as shown in Figure 3-3. Without traffic loading a confining stress, caused by overburden, is applied to the element. As a wheel load approaches the element, the element is subjected to a simultaneous build-up in both the major principal and minor principal stresses. These stresses also rotate about the centre of the element as shown in Figure 3-3 and this is called the rotation of principal stresses.

If the element is not rotated (also shown in Figure 3-3) then, as the load approaches, the vertical stress and the horizontal stress increase. The shear stresses, increase as the load approaches to a point where they start to decrease until the load is directly above the particular element at which point there is no shear stress on vertical and horizontal planes. This is when pure triaxial conditions exist. As the load moves away a complete reversal of shear stress occurs, this too is shown in Figure 3-3. Unfortunately, as discussed later, these stress rotation effects cannot be duplicated in the repeated load triaxial apparatus and this is therefore a limitation of this apparatus.

Figure 3-3 Loading of an Element in a Pavement Showing the Rotation of the Principal Stresses.



3.2.2 Vehicular Loading Characteristics

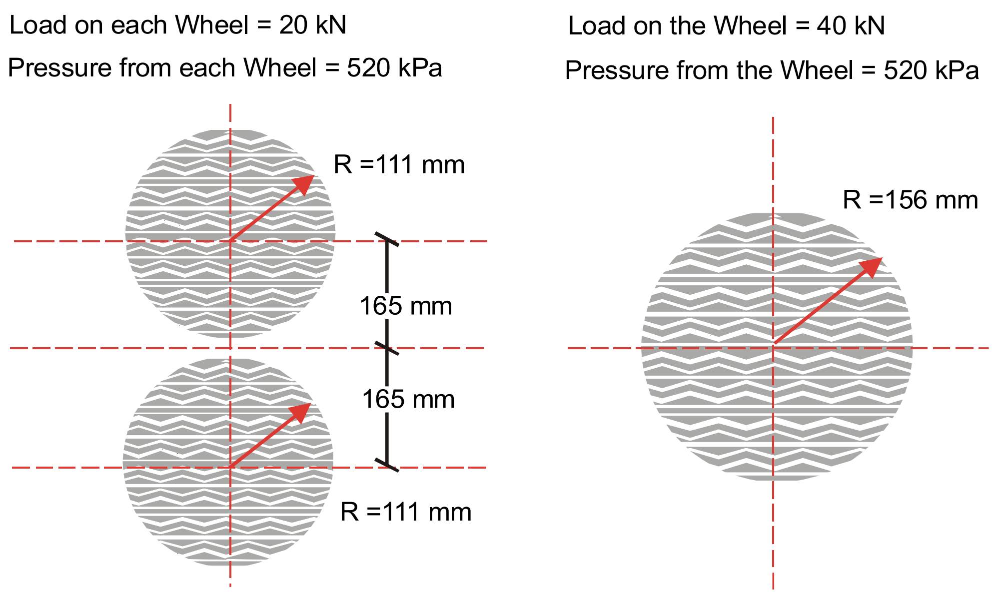
Wheeled vehicular loading causes cyclic stresses to be applied to the pavement structure. These are generally in the form of a large number of rapidly applied stress pulses of varying magnitude. For pavement design purposes it is necessary not only to consider the total number of wheel loads but also the magnitude of individual loads, their duration and the frequency of loading repetition.

<u>Loading Magnitude</u>

Each wheel load will be of a different magnitude. Since the damage caused by each wheel passage on the pavement depends strongly on this magnitude it is convenient, and practically necessary, to equate the applied wheel loads over the life of the road pavement to an equivalent number of standard wheel loads that would cause the same damage {Paterson (1987)}. This is achieved by expressing the cumulative load in terms of Equivalent Standard Axles (ESA). Where it is desirable to express pavement life in terms of years, rather than ESAs, it is common to relate the traffic pattern expected to the ESAs that they will produce by classifying the vehicles into various groups each with a set number of ESAs per vehicle.

Most pavement designs are based on a standard 80 kN axle load comprising four tyres, two on each side of the axle. Some computer analysis programs allow the application of multiple loads and this actual configuration can be used, unfortunately however, some computer programs only allow for a single load. Consequently, two different load configurations are used in this work, for the analytical procedures, each applying the same pressure to the road surface as defined in Figure 3-4.

Figure 3-4 Pavement Loading Characteristics

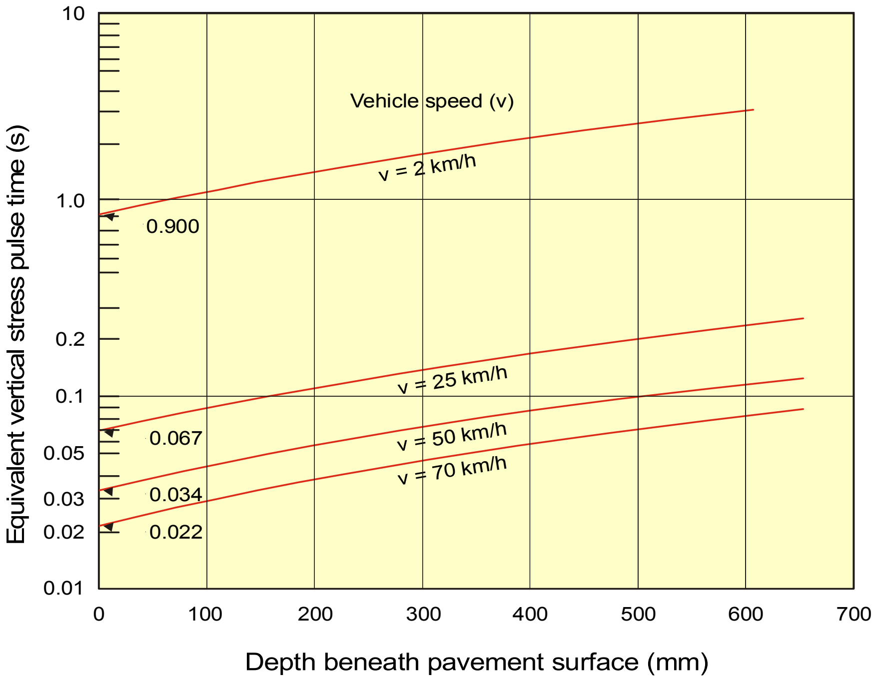


Load Duration and the Frequency of Loading Repetition

The duration of the load in a road pavement has been investigated {Barksdale (1971)}. The time at which a vertical stress pulse is applied to a pavement, for different vehicle speeds at different depths, is shown in Figure 3-5.

The frequency of loading repetition is controlled by the speed at which vehicles travel on the road, the type of vehicle using the road and the number of vehicles on the road. Therefore in order to simulate traffic moving over a pavement surface, it is necessary to apply a large number of rapidly applied stress pulses. For moderate vehicle speeds, the stress pulse lasts between about 0.02 and 0.2 sec. The pulse time increases with increasing depth in the pavement and decreasing vehicle speed.

Figure 3-5 Vertical Stress Pulse Time as a Function of the Depth in a Pavement for different Vehicle Speeds



After Barksdale (1971)

The vehicle speed determines the rate of loading. For this reason vehicle speed needs to be taken when considering the properties of bituminous mixtures due to their visco-elastic behaviour. The rate of loading, therefore, directly affects the vertical deflection under loading. Other materials, such as fine-grained soils, may also be sensitive to rate of loading although to a lesser extent, but these are usually located so deep in the pavement that the vehicle speed has a relative minor effect on their properties.

It was shown by Barksdale et al.(1990) that near the surface the stress pulse has a pronounced haversine shape and with depth the pulse duration becomes greater and, although it remains haversine in shape, a triangular loading gives a reasonably good approximation.

3.3 THE BEHAVIOUR OF PAVEMENT MATERIALS UNDER TRAFFIC LOADING

A better understanding of the behaviour of the materials that are used for road construction will allow materials to be used more economically and with more environmental acceptance. This detailed understanding will enable more relevant specifications to be compiled rather than those defined by the present empirical specifications.

3.3.1 Subgrade Soils

With regard to fine grained soils, or soils whose behaviour depends on the fine fraction there is limited knowledge on the quantitative influence on mechanical properties of:

- The water content, which results from environmental and drainage conditions;
- The stress history of the soil, this is especially the case for re-compacted soils which form the pavement's foundation as embankments (and probably elsewhere).

Such influences are not rationally accounted for in empirical design methods.

Previous work on soils has indicated that the elastic parameters of soils are influenced by a number of variables {Seed at al (1955), Hyde (1974), Loach (1987)}. These include not only the physical conditions such as the stress state, moisture content, and the pre-consolidation pressure but also such variables as the rate of loading and number of load applications.

The stress-strain response of soils is known to be non-linear. The sensitivity of the life of a pavement to changes in the value of Young's Modulus of the subgrade was demonstrated by Hight and Stevens (1982) and thus there is a clear need to accurately estimate the modulus of the subgrade at the design stage. The elastic properties are needed so that the resilient modulus of the foundation under construction or under normal traffic loading can be computed and an appropriate design of the placed layers, especially the bound layers, undertaken.

Conventionally, subgrades are characterised by a CBR, either determined *in-situ* or in the laboratory. As discussed in the previous chapter, this is an empirical penetration test (principally inducing failure) and is not of direct use in an analytical design method, which requires a stress-strain model. A further difficulty is the need to test the soil at those stresses, water and suction conditions that may pertain during the life of the pavement, a condition that may differ from those at the time of design.

These different conditions can be controlled and varied during a repeated load triaxial test. The resilient and permanent strains resulting from different repeated stress paths and different states of initial suction in soils was a principal area of study of the 'Science Project' on which this thesis draws. For the work in this thesis material characteristics such as moisture content and stress application values were chosen as close to the actual values found in road pavements in Europe as was possible.

Since these materials are fine grained, small specimens can be tested thus allowing more conventional triaxial apparatus to be modified for repeated load testing.

3.3.2 Unbound Granular Materials

In recent years much work has been done on the investigation of the behavioural properties of unbound granular materials and subgrade soils in pavement structures under traffic loading {Sweere (1990), Boyce (1976), Pappin (1979)}. Unbound granular materials generally make up the subbase layers of roads and may be used in base layers of relatively lightly trafficked roads. These materials are generally characterised by their resistance to impact, for example Los Angeles Abrasion value, and their geological characterisation. This means that marginal materials that might have been satisfactory for construction as well as being cheaper are often excluded by specifications. Since these granular materials are usually found higher in the pavement they are subjected to higher levels of stress and more markedly rotating stresses. For this reason more detailed characterisation by advanced test methods may be more important than for the materials found lower down in the pavement structure (subgrade soils).

Some research has been conducted {Cheung (1994)} to find an inexpensive simple test that allows these materials to be characterised with respect to their stress-strain response, but it appears that the repeated load triaxial test remains the most appropriate test method.

For the study of full sized aggregates used in road pavements, railway ballast or other engineering applications, such as earthquake problems, where repeated loading occurs, the specimen size must be large enough so as not to have an influence on the behaviour of the material. A general rule of thumb is that the diameter of the specimen should be not less than ten times the largest particle size, although often seven is used. Thus for road aggregates of 0/40 mm grading a specimen diameter of about 300 mm is required.

3.4 REPEATED LOAD TRIAXIAL TESTING

The principle of repeated load triaxial tests is to simulate an element in a pavement by manufacturing a specimen of road construction material and applying similar load conditions to those that might be experienced in the field while measuring the deformation experienced by the specimen. The purpose of the test is to determine the material parameters under simulated traffic loading conditions, such as resilient strain parameters and permanent strain parameters. Due to the stress dependent behaviour of these unbound materials the elastic parameters must be determined at a number of stress levels.

A solid cylindrical specimen is formed by compacting material into a steel mould at a predefined moisture content and density. The specimen is placed on top of a rigid bottom platen and a second rigid platen is then placed on top of the specimen. The specimen is sealed against the platens by enclosing it within a rubber membrane. The specimen is then either placed in a cell where an all-round confining stress is applied to the specimen or the confining pressure is applied by means of a partial vacuum within the membrane. This confining pressure simulates the lateral stress caused by the overburden pressure. An axial load through the platens simulates the applied wheel loadings.

The triaxial compression test has been used as the basic testing apparatus in geotechnical engineering to evaluate stiffness and shear strength of cohesive and granular materials. The early pioneering work performed by Seed et al (1955); Seed and Fead (1959), and Seed at al (1962) was all conducted using the repeated load triaxial test apparatus. This apparatus was first developed from the monotonic load triaxial test by incorporating loading systems that could simply apply and remove the deviator stress, while the confining stress was kept constant. However, Allen and Thompson (1974) reported going one step further by applying not only a repeated deviator stress, but also a repeated confining stress. These Variable Confining Pressure (VCP) tests are a closer simulation of actual field conditions than the Constant Confining Pressure (CCP) tests, since in the road structure the confining stress acting on the material is repeated as a load approaches.

Today the repeated load triaxial compression test is by far the most commonly used method to evaluate the resilient modulus for pavement design and research purposes {Barksdale et al (1990)}.

3.4.1 Repeated Load Triaxial Apparatus Configurations

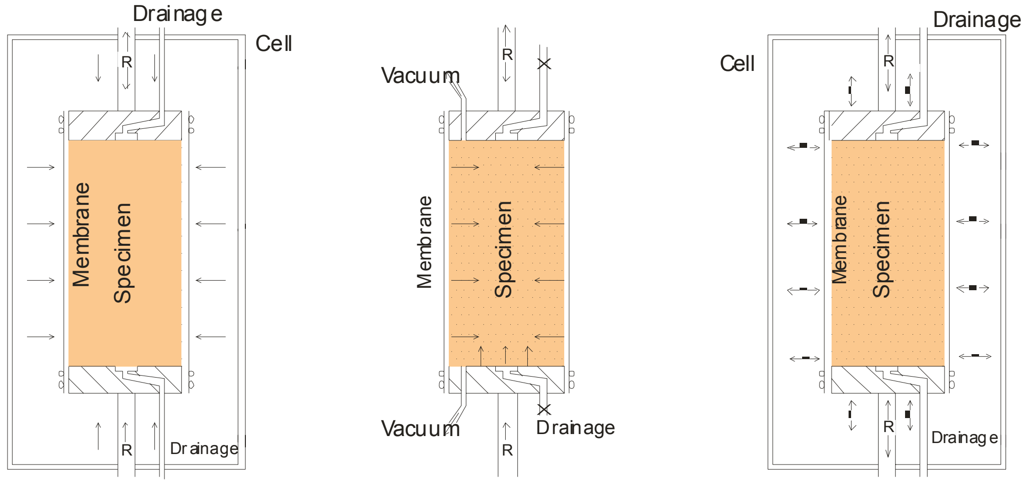
There are basically three different methods, and corresponding apparatus, with which one can conduct repeated load triaxial tests on road construction materials. These different configurations are shown schematically in Figure 3-6 and illustrated in Photograph 3-1 and are as follows:

- Conventional repeated load triaxial test, with constant confining pressure.
- Internal vacuum repeated load triaxial test, with constant confining pressure.
- Variable confining pressure repeated load triaxial test.

<u>Conventional Repeated Load Triaxial Test</u>

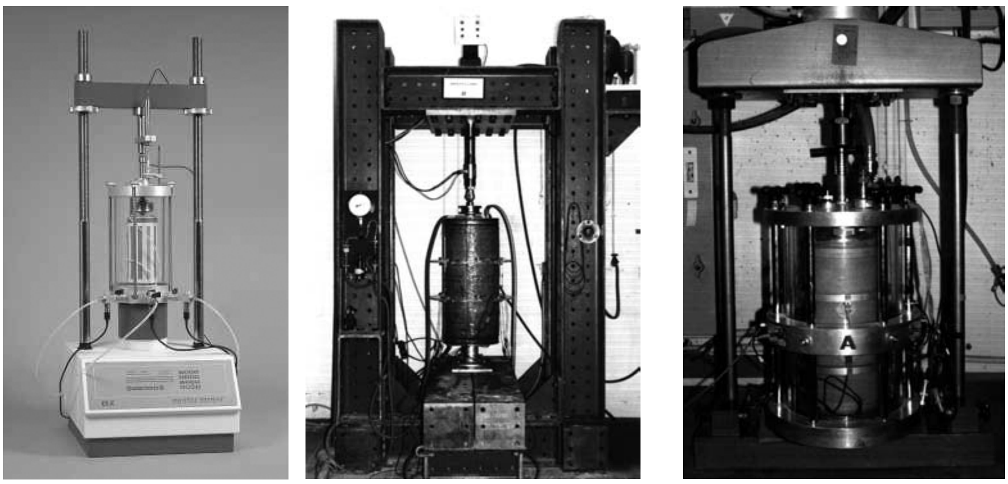
The cylindrical specimen is placed inside a triaxial cell on top of a rigid bottom platen, a rigid platen is placed on top of the specimen, and a rubber membrane is placed around the specimen. A cylindrical chamber is placed around the specimen. The specimen is subjected to a constant confining pressure by increasing the pressure within the chamber. Drainage lines lead to porous stones located in the top and bottom platens and atmospheric pressure can be maintained within the specimen by leaving these lines open. By keeping the pressure inside the specimen at atmospheric pressure the confining pressure applied to the specimen is equal to the pressure applied within the cell (measured relative to atmospheric pressure). A repeated axial load is applied to the ends of the specimen by way of the platens simulating traffic loading.

Figure 3-6 Schematic Illustration of the Repeated Load Triaxial Apparatus



Conventional repeated load triaxial test. Vacuum method of repeated load triaxial test. Variable confining pressure repeated load triaxial test.

Photograph 3-1 Repeated Load Triaxial Apparatus



Conventional repeated load triaxial apparatus Vacuum method of repeated load triaxial method Variable confining pressure repeated load triaxial method

Vacuum Method Repeated Load Triaxial Test

This test is similar to the conventional test except that, instead of the confining pressure being applied by fluid within a cell surrounding the specimen, the confining pressure is supplied by means of a partial vacuum inside the membrane. This method is used on big specimens of granular material where, due to the size of the specimen, the cell would be impracticably large. Again, a repeated axial load is applied to the ends of the specimen simulating traffic loading.

Variable Confining Pressure Repeated Load Triaxial Test

In reality, the lateral pressure on an element of material in a pavement builds up as vehicles approach and decreases as they move away. Thus it is desirable to vary the confining pressure by using fluid in the cell controlled by a fairly sophisticated electronic control system. A repeated load is applied axially as above and simultaneously a repeated confining load is applied to the specimen via the fluid. These loads are independent of one another and therefore many stress regimes may be applied.

The repeated triaxial test offers four very important advantages in the investigation of the elastic properties of materials:

1.	Stress state	Known principal stresses are applied to the specimen in known directions. Thus the stress conditions within the specimen on any plane are also known throughout the test.
2.	Strain measurements	Axial and radial and thus volumetric and shear strains can be measured and the permanent and resilient deformation calculated. Young's modulus (or resilient modulus) Poison's ratio and the bulk and shear moduli can be determined.
3.	Suction measurements	Pore pressures can be easily measured at the ends of the specimen or, with more difficulty, within the specimen.
4.	Specimen drainage	The triaxial test allows relatively simple, controlled drainage of the specimen in the axial and radial directions.

The most significant disadvantage of the triaxial cell is its limited ability to simulate the rotation of the principal stress axes and the shear stress reversal. Only fixed orthogonal rotation of principal stress axes are possible in this test. Also, the intermediate principal stress applied to a specimen cannot be independently controlled in the triaxial test.

3.4.2 Variable Confining Pressure versus Constant Confining Pressure

Although the introduction of VCP has improved the accuracy of the simulation of the actual loading in the pavements, this has resulted in the test apparatus becoming more complex and thus more prone to errors. However, the advent of cheaper sophisticated electronic control equipment is easing the complexity somewhat.

Allen and Thompson (1974) investigated the influence of repeating the confining pressure on the elastic parameters (resilient modulus and Poisson's ratio) of granular materials. They compared variable and constant confining pressure test results on similar specimens and reported that higher values were obtained for the elastic parameters in the CCP tests those in the VCP tests.

Brown and Hyde (1975) later showed that two different confining pressure tests yielded the same resilient modulus values, provided the confining stress in the CCP test was equal to the mean value of confining stress in the VCP test. Since Allen and Thompson had used the peak value from their VCP tests in their CCP tests, the higher values for resilient modulus found in the CCP tests were concluded to be attributable to the higher stress level in the test material.

Where the VCP and CCP tests yielded the same resilient modulus values, Brown and Hyde (1975) showed that the Poisson's ratio in the CCP test differed considerably from those obtained in VCP tests. The stress dependency of Poisson's ratio found in the two types of tests was completely opposite. VCP tests yielded decreasing Poisson's ratio values for increasing ratios of deviator stress over confining stress, whereas in CCP tests Poisson's ratio was found to increase with increasing stress ratio. Values of Poisson's ratio over 0.5 were found in the CCP tests, thereby indicating resilient specimen dilation or volume increase.

Using a more fundamental approach to stress strain relationships, Brown and Hyde (1975) showed that the problem of deviating values for Poisson's ratio found in VCP and CCP tests can be circumvented. Separating stresses and strains into volumetric and shear components, they showed that VCP and CCP tests do yield the same stress strain relationships for those stress ratios that do not cause specimen dilation. The separation of stresses and strains into volumetric and shear components will be discussed in more detail in a forthcoming chapter. Brown and Hyde obtained a reasonable correlation between VCP and CCP tests with respect to permanent strain again, by setting the confining stress in the CCP tests at the mean value of confining stress in the VCP tests.

The confinement method is largely dependent on the material particle size. Since crushed rock granular bases comprise coarse particles of up to 40 mm, strictly, these materials require large specimens of up to 400 mm in diameter and 800 mm in height. Sweere (1990) stated that it is impractical to construct apparatus with confining cells that can accommodate such large specimens and concluded that CCP tests are adequate for determining material parameters for granular materials provided the appropriate models are used to relate strains to stresses.

3.4.3 Apparatus Produced Factors that Influence the Triaxial Test Results

The elastic parameters of unbound materials are particularly sensitive to the testing equipment used and the test procedure {Barksdale et al (1990)}. Therefore, particular attention must be paid to equipment and procedure details in order to obtain reliable values for the elastic parameters of the materials.

Specimen Alignment

Errors due to poor alignment of the specimen in the apparatus, causing the axis of the applied load not to coincide with the axis of the specimen, have been shown to result in seriously erroneous results {Moore et al (1970)}. Misalignment will induce specimen ‘bending’ causing non-uniform load distribution and consequent non-uniform strain on different sides of a specimen. Similar erroneous stresses in the specimen are found as a consequence of the ends of a triaxial specimen not being perpendicular to the long axis of the cylindrical specimen. To minimize misalignment errors the triaxial cell must be carefully machined to be in perfect alignment, the cell must be aligned relative to the external loading ram, the specimen must be a right circular cylinder and the specimen must be aligned with the triaxial cell.

Seating Errors and Bedding Errors

Seating errors are unwanted deformations that occur when two surfaces of the testing apparatus do not marry up perfectly against one another. Seating errors occur at a number of critical places within the apparatus such as the connection between the load ram and the top load platen, between the bottom platen and the base of the triaxial cell, and between the loading platens and the porous stones.

During the manufacture of specimens and their installation into the apparatus small irregularities exist between the ends of the specimen and the apparatus platens. As a result of this non-uniform contact, unwanted deformation occurs in the vicinity of the specimen ends. Many researchers have reported the seriousness of bedding errors {Baladi et al (1988); Clayton and Khatrush (1987); Burland and Symes (1982)}, although Burland and Symes state that bedding errors are really only significant in static tests at low deviator stress levels. During repeated load triaxial tests bedding errors are assumed to be insignificantly small, provided that an application of load cycles during the preconditioning phase is made.

However, it has been shown that seating and bedding errors may still be present after preconditioning. Pezo et al (1991) reported that strong contact between the specimen and top and bottom platens is very important. Poor seating of the platens may introduce about 20% error in resilient modulus determination. Pezo et al (1991) used hydrostone paste between the test specimen and the end caps, which provided uniform contact between the specimen and caps thus eliminating some of the error in measuring sample deformations.

End Friction or Restraint

End friction is due to the friction between the specimen end and the end platen. It can never be completely eliminated and can result in the specimen assuming a barrel shape when loaded. This causes a non-uniform strain distribution down the axis of the specimen. Early static tests by Rowe and Barden (1964) show that polished end platens coated with silicone grease and the insertion of a latex rubber disk between the specimen and the end platens help to minimise friction. Boyce (1976) concluded that better performance is obtained by replacing the rubber disc by a stainless steel disc cut into numerous segments. He showed that during repeated load tests performed on granular material barrelling did not occur when using frictionless ends.

Barrelling of the specimen concentrates the lateral strain in the middle {Boyce (1976)}. Therefore greater deformation would be measured at the middle of the specimen rather than the ends and this must be considered when applying measurement apparatus to the specimens. Dehlen (1969) concluded, however, end friction is not too important in resilient modulus testing provided the specimen height to diameter ratio is at least two.

It has been reported that the most successful method of reducing end friction is to use high vacuum silicone grease on polished steel end platens with a rubber membrane separating the soil and grease {Lee (1976), Brown (1974), Overy (1982)}. However, because some end restraint will always be present, the measured pore pressure will not be exactly correct. In order to minimise this error Sangrey et al (1969) applied very low frequencies of loading, which allow time for pore pressure equalisation. While Koutsoftas (1978) allowed time after faster cyclic loading for the pore pressure to equalise. In either case, the end effects will distort the recorded pore pressure but the second method has the advantage of testing at representative frequencies or rates of loading. Loach (1987), testing subgrade soils, stated that the most accurate pore pressure measurements would be from a centre probe in the relatively uniform central section of the sample before pore pressure equalisation has taken place.

System Compliance

When axial deformations are measured outside of the triaxial cell, they include both the deformation of the specimen as well as all other deformations that occur between the point where deformation is measured and the fixed reference point that does not move. System compliance is the deformation that occurs within various parts of the triaxial cell, load cell, and support system. Compliance is present in all components of the apparatus, particularly internally mounted load cells, porous stones, filter paper, end platens and frictionless ends {Clayton and Khatrush (1987)}. In some instances compliance may even occur in the system used to support the apparatus.

Clayton and Khatrush (1987) showed that it is possible to quantify these errors by calibrating the triaxial apparatus using a dummy specimen, in which the deformation can be determined accurately under a given level of loading. Aluminium and steel is frequently used for the dummy since these materials have a known modulus of elasticity.

<u>Deformation Measurement</u>

Axial resilient deformation measurements during repeated load testing have been made both outside {Parr (1972); Lashine (1971); Barksdale (1972b)} and inside the triaxial cell {Terrel (1967); Dehlen (1969); Hicks (1970); Barksdale (1971); and Crockford, et al., (1990)}. As a result of end effects, the strain and stress distribution is not uniform within the specimen. To avoid this problem and also those of bedding, seating, and system compliance, axial deformation measurements should be measured on the specimen.

In order to alleviate the end effects and other errors it is common to measure the deformation of the specimen at either the $^1/_4$ points in from each end of the specimen {Boyce (1976); Allen and Thompson (1971); Hicks and Monismith (1971); Barksdale, (1972b)} or at the $^1/_3$ points as reported by Chisolm and Townsend (1976). The advantage of the larger gauge length is that larger deformations are experienced that can be more accurately recorded. The disadvantage, however, is that the radial deformation should be measured as close as possible to the centre of the specimen, where the strain distribution is reasonably uniform.

After preparing the specimen, Boyce and Brown (1976) placed four small LVDT's between 4 pairs of studs to measure axial strain in a limestone base. They concluded that the large aggregate present caused considerable variation in strain from one location to another. Also, Boyce and Brown concluded that measurement of at least three axial strains is necessary to provide a reliable average value.

When measuring both the radial and the axial deformation at specific points on a specimen, for reasons of basic geometry it is desirable to measure movement at three points since this gives sufficient deflection data to define the complete plane of the strain within the specimen.

3.4.4 Triaxial Stress State

The strength of an unbound material (granular or soil) is expressed in terms of the maximum shear stress that it can sustain under given conditions. This strength depends on the friction and interlock that are mobilised between particles. The shear strength may therefore, be expressed as:

$$\tau = c' + \sigma' \tan \varphi' \qquad \text{Eqn.3-1}$$

Where	τ	shear strength
	c'	cohesion
	Φ'	angle of shearing resistance

This equation can be regarded as somewhat analogous to an angle of friction in classical mechanics. The shear strength depends on the state of compaction of the material (higher values being associated with dense packing) and the levels of deformation involved.

<u>Principal Stresses</u>

The element within a pavement structure, as described above, experiences a stress pulse, caused by the loading from a passing vehicle as well as the constant overburden stress. This stress pulse has three components:

i) Vertical compressive stress (σ_v).

ii) Horizontal, stress (σ_h), normally compressive but may be tensile at the bottom of stiff bound layers.

iii) Shear stresses (τ_{vh}, τ_{hv}), which are reversed as the load passes as a consequence of the rotation of the planes of principal stress.

If one were to rotate the element, namely the three orthogonal planes, in such a way that there are zero shear stresses acting on the element, then the normal stresses that act on these planes are called the principal stresses. The largest of these three stresses is called the major principal stress, the smallest is called the minor principal stress, and the third one is called the intermediate principal stress. These are denoted by (σ_1, σ_2, σ_3) respectively.

In a pavement situation it is only necessary to consider the state of stress in the plane that contains the major and minor principal stresses.

In soil mechanics, it is usual to take compressive direct stresses and deformations, and anticlockwise shear stresses and the associated shear deformations, as positive. This is in contrast to structural mechanics, in which tensile direct stresses and deformations, and clockwise shear stresses, are conventionally taken as positive.

In the specific case of a pavement structure with a static load caused by the overburden and dynamic loading, brought about by an approaching single wheel load, the horizontal stresses on the element are only equal when the wheel load is directly above the element. At this point the shear stresses are zero. Thus, this is the only time during the loading that the triaxial apparatus simulates the exact conditions on a specimen as compared to those on the element in the pavement. All other situations, including loading from dual wheel loads, cannot be reproduced in the triaxial apparatus. In the situation where a single wheel load is directly above the element, and in the triaxial apparatus, the following stress state exists:

- The vertical stress equals the major principal stress ($\sigma_v = \sigma_1$);
- The horizontal stress equals the minor and intermediate principal stresses ($\sigma_h = \sigma_2 = \sigma_3$), and;
- The shear stress is zero $\tau_{vh} = \tau_{hv} =0$.

Therefore, the repeated triaxial apparatus can be imagined to be applying stresses to the element from directly above the element but with varying load strength simulating the vertical and horizontal stress applied to the element. This is a simplification, but an acceptable one considering the complexities of apparatus that are capable of exactly the correct loading conditions {Thom (1988), Chan (1990)}.

Given the magnitude and direction of the principal stresses it is possible to compute normal and shear stresses in any other direction using the following equations:

$$\sigma_\theta = \sigma_1 \cos^2\theta + \sigma_3 \sin^2\theta = \frac{\sigma_1 + \sigma_2}{2} + \frac{\sigma_1 - \sigma_2}{2}\cos 2\theta$$

Eqn.3-2

$$\tau_\theta = (\sigma_1 - \sigma_3)\sin\theta\cos\theta = \frac{\sigma_1 + \sigma_2}{2}\sin 2\theta$$

These equations provide a complete two-dimensional description for the state of stress and describe a circle, known as the Möhr circle. Any point on the circle represents the stress on a plane whose normal is orientated at an angle (ϴ) to the direction of the major principal stress and the maximum shear stress equals the radius of the circle.

<u>Stress Invariants</u>

A physical interpretation of a three-dimensional stress system is obtained by considering the applied stresses to be divided into those stresses that tend to cause volume change (mean normal stress) and those that cause shear distortion (shear stress). In practice, shear stress may not only cause shear distortion but also volumetric dilation or contraction and vice versa for all round stress.

The mean normal stress is a measure of the stresses that cause volume change, and is defined as:

$$p = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3) \qquad \text{Eqn.3-3}$$

Where: p mean normal stress

The octahedral shear stress is a measure of the shear distortion of the material, and is defined as:

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \qquad \text{Eqn.3-4}$$

Where: τ_{oct} octahedral shear stress

These two parameters are called invariants since they are independent of direction. Assuming the axial symmetry under a wheel load, as discussed above in a pavement situation, the horizontal stresses are taken as equal and considering that the materials will be either partially saturated or saturated, the normal and shear stress invariants can be written as follows:

$$p' = \frac{1}{3}(\sigma'_1 + 2\sigma'_3) \qquad \text{Eqn.3-5}$$

$$\tau'_{oct} = \frac{\sqrt{2}}{3}(\sigma'_1 - \sigma'_3) \qquad \text{Eqn.3-6}$$

Where the prime (‘) indicates that the parameter is effective, rather that total (discussed in the next chapter).

In a conventional triaxial test the deviator stress is defined as:

$$q = \sigma_1 - \sigma_3 \qquad \text{Eqn.3-7}$$

Where: q deviator stress

Again, due to partially saturated or saturated conditions, and in keeping with standard soil mechanics practice in a triaxial situation:

$$q = (\sigma_1 - \sigma_3) = (\sigma'_1 - \sigma'_3) = \frac{3}{\sqrt{2}}\tau'_{oct} \qquad \text{Eqn.3-8}$$

Therefore of the two invariants used mean normal effective stress is affected by pore pressure while the deviator stress is not affected by pore pressure.

<u>Strain Invariants</u>

Strain is defined as the deformation per unit of original length, and is dimensionless. It is often reported in microstrain where 1 microstrain is defined as 1 millionth of the original length. It was shown in Figure 3-2 above that with each load cycle there exists an elastic deformation that recovers after each load and a small permanent deformation which in irrecoverable.

Resilient strain (ε_r) is defined by:

$$\varepsilon_r = \frac{\Delta L_{(N)}}{L_0(1-\varepsilon_{p(N-1)})} \qquad \text{Eqn.3-9}$$

Permanent strain (ε_p) is defined as:

$$\varepsilon_p = \frac{\Delta L_{(Total)}}{L_0} \qquad \text{Eqn.3-10}$$

Where:	L_0	original specimen length (height or diameter)
	$\Delta L_{(Total)}$	total plastic change in specimen length
	$\Delta L_{(N)}$	resilient change in specimen length for N cycles
	N	number of cycles

Strains may also be translated into their appropriate invariants using the same approach that was used for stresses. The mean normal stress tends to cause volume change, which has a corresponding strain invariant called volumetric strain and is defined as:

$$\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 \qquad \text{Eqn.3-11}$$

Where ε_v volumetric strain

The octahedral shear stress tends to cause a shear strain and is defined as:

$$\varepsilon_s = \frac{2}{3}\sqrt{(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2} \qquad \text{Eqn.3-12}$$

Where ε_s shear strain

Again due to the assumed axial symmetry under a wheel load in the pavement situation, the horizontal stresses, and thus strains, are considered equal resulting in the volumetric and shear strains to be simplified as:

$$\varepsilon_v = \varepsilon_1 + 2\,\varepsilon_3 \qquad \text{Eqn.3-13}$$

$$\varepsilon_s = \frac{2}{3}(\varepsilon_1 - \varepsilon_3) \qquad \text{Eqn.3-14}$$

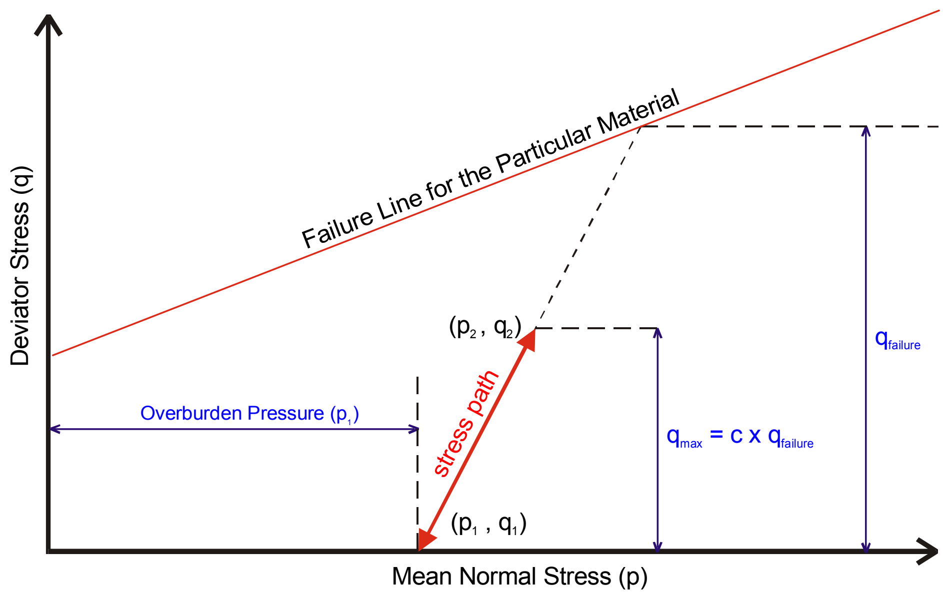
In summary, by taking an element in a pavement structure and applying the assumed traffic loading conditions, the mean normal stress applied on the element tends to cause volumetric deformation, which is monitored as volumetric strain. Also, a deviator stress applied to the element tends to cause shear deformation, which is monitored by shear strain. However, there is also cross-coupling since shear stress causes dilative or contractive volumetric stresses and due to the volumetric stress change shear stresses are generated.

Stress Paths and p-q Diagrams

It is often desirable to depict the successive states of stress that exist in material as the specimen is loaded and unloaded. The accepted method of showing this is to plot a series of stress points {Boyce (1976)}.

These stress points have co-ordinates namely mean normal stress and deviator stress and if these points are connected with a line or a curve. This curve is called a stress path. By varying the stress path applied to the specimen a large number of the stress regimes may be investigated. A sensible test approach is to load the specimen along predetermined stress paths to simulate traffic loading on an element in a pavement structure. A stress path, therefore, gives a continuous representation of successive states of stress. For this work certain points are defined on a stress path, such as the start and end points. These are illustrated in Figure 3-7. Note the material failure line is also shown in this figure. This failure line is defined by conducting strength tests in a standard triaxial apparatus.

Figure 3-7 The Definition of a Stress Path in p-q Space



Possible Stress Path Regimes

During a repeated load triaxial test it is required that the vertical (σ_v) and horizontal (σ_h) stresses applied to the specimen are greater than or equal to zero. Two stress regimes can be applied to the specimen, namely:

- Compression, where $\sigma_v > \sigma_h$ and $q > 0$, or;
- Extension, where $\sigma_v < \sigma_h$ and $q < 0$.

With reference to Figure 3-8,

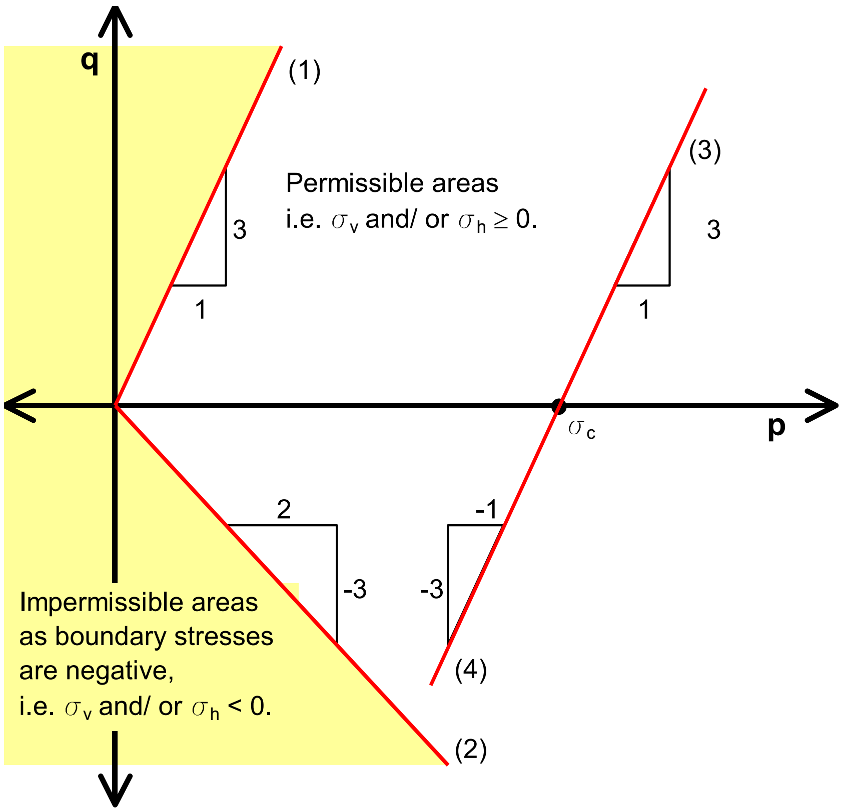
Taking the cell pressure (σ_c) to be zero for the compression stress regime:

$\sigma_h = 0$, $\sigma_v > 0$ and therefore $\Delta\sigma_v > 0$, so

$\Delta q = \Delta\sigma_v$ and $\Delta p = \Delta\sigma_v/3$, therefore:

$\Delta q/\Delta p = 3/1$ shown as stress path (1).

Figure 3-8 Possible Stress Regimes in a Repeated Loads Triaxial Test



Taking the cell pressure (σ_c) to be zero for the extension stress regime:

$\sigma_h = 0$, $\sigma_v > 0$ and therefore $\Delta\sigma_v > 0$, as the cell pressure is increased

$\Delta\sigma_c = \Delta\sigma_h$, which also increases $\Delta\sigma_v$, therefore it is necessary to reduce $\Delta\sigma_v$ by $\Delta\sigma_c$ so $\Delta\sigma_v = 0$,

so $\Delta q = \Delta\sigma_v - \Delta\sigma_h = -\Delta\sigma_c$, and $\Delta p = (\Delta\sigma_v + 2\Delta\sigma_h)/3 = 2\Delta\sigma_c/3$, therefore:

$\Delta q/\Delta p = -\Delta\sigma_c/(2\Delta\sigma_c/3) = -3/2$ shown as stress path (2).

Taking the cell pressure (σ_c) greater than zero for the compression stress regime:

$\sigma_h > 0$, $\sigma_v > \sigma_h > 0$ and therefore $\Delta\sigma_v > 0$, so

$\Delta q = \Delta\sigma_v - \Delta\sigma_h = \Delta\sigma_v$ and $\Delta p = (\Delta\sigma_v + 2\Delta\sigma_h)/3 = \Delta\sigma_v/3$, therefore

$\Delta q/\Delta p = 3/1$ shown as stress path (3).

Taking the cell pressure (σ_c) greater than zero for the extension stress regime:

$\sigma_h > 0$, $\sigma_h > \sigma_v > 0$ and therefore $\Delta\sigma_v < 0$, so

$\Delta q = \Delta\sigma_v - \Delta\sigma_h = -\Delta\sigma_v$ and $\Delta p = (-\Delta\sigma_v + 2\Delta\sigma_h)/3 = -\Delta\sigma_v/3$, therefore

$\Delta q/\Delta p = 3/1$ shown as stress path (4).

3.4.5 Resilient Modulus and Poisson's Ratio

In order to characterise unbound materials there are two important material parameters, namely resilient modulus and Poisson's ratio. Both of these parameters are stress dependent, but assume linear elastic behaviour.

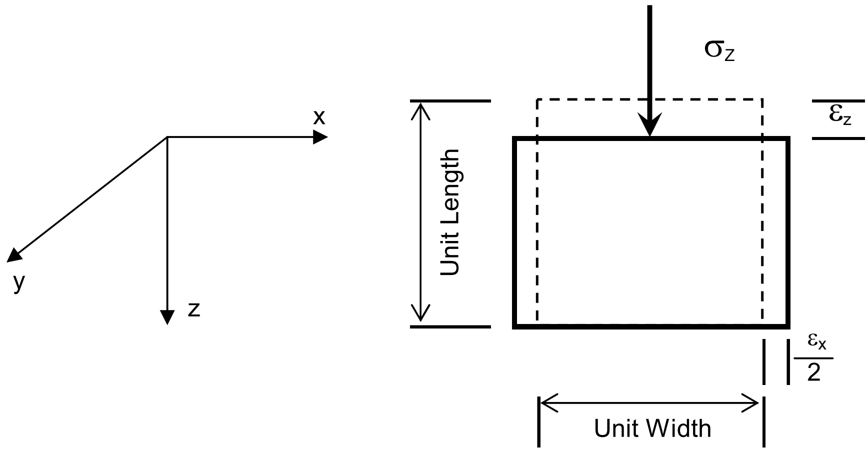
Hooke's Law

These parameters are defined using Hooke's Law. Hooke's law describes the linear relationship between stress and strain for a uniaxial stress condition as shown in Figure 3-9 and defined by:

$$E = \frac{\sigma_z}{\varepsilon_z} \qquad \text{Eqn.3-15}$$

Where E Young's modulus

Figure 3-9 Uniaxial Stress Condition Hooke's Law



Hooke's law for the uniaxial case can be expanded to deal with a triaxial stress condition where normal stresses act in x, y and z direction by superposing the strains obtained from the above equations. The equations obtained are known as the 'generalized Hooke's law':

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_3 \end{Bmatrix} = \frac{1}{E}\begin{bmatrix} 1 & -v & -v \\ -v & 1 & -v \\ -v & -v & 1 \end{bmatrix}\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \end{Bmatrix} \qquad \text{Eqn.3-16}$$

Where $\varepsilon_1; \varepsilon_2; \varepsilon_3$ Normal strains
$\sigma_1; \sigma_2; \sigma_3$ Normal stresses

For the axisymmetric stress condition of the repeated load triaxial test, stresses and strains in x and y direction are equal and these equations reduce and are the equations that should be used to interpreted repeated load triaxial test results:

$$\varepsilon_1 = \frac{1}{E}\left[\sigma_1 - 2v\sigma_3\right]$$

Eqn.3-17

$$\varepsilon_3 = \frac{1}{E}\left[-v\sigma_1 + (1-v)\sigma_3\right]$$

By replacing M_r for E the first of the equations in Equation 3-17 the equation can be written as follows:

$$M_r\varepsilon_1 = \sigma_1 - 2v\sigma_3$$

Eqn.3-18

$$v = \frac{\sigma_1 - M_r\varepsilon_3}{2\sigma_3}$$

and then substituting this equation in the second of the equations in Equation 3-17:

$$M_r = \frac{(1-v)\sigma_3 - v\sigma_1}{\varepsilon_3} \qquad \text{Eqn.3-19}$$

$$M_r = \left(\frac{2\sigma_3 - \sigma_1 + M_r\varepsilon_1}{2\varepsilon_3\sigma_3}\right)\sigma_3 - \left(\frac{\sigma_1 - M_r\varepsilon_1}{2\varepsilon_3\sigma_3}\right)\sigma_1 \qquad \text{Eqn.3-20}$$

$$M_r = \frac{2\sigma_3^{\,2} - \sigma_1\sigma_3 + M_r\varepsilon_3\varepsilon_1 - \sigma_1^{\,2} + M_r\sigma_3\varepsilon_1}{2\varepsilon_3\sigma_3} \qquad \text{Eqn.3-21}$$

$$M_r\left[1 - \frac{(\sigma_1 + \sigma_3)\varepsilon_1}{2\varepsilon_3\sigma_3}\right] = \frac{2\sigma_3^{\,2} - \sigma_1\sigma_3 - \sigma_1^{\,2}}{2\varepsilon_3\sigma_3} \qquad \text{Eqn.3-22}$$

$$M_r = \frac{2\sigma_3^{\,2} - \sigma_1\sigma_3 - \sigma_1^{\,2}}{2\varepsilon_3\sigma_3 - (\sigma_1 + \sigma_3)\varepsilon_1} \qquad \text{Eqn.3-23}$$

Multiplying the top and bottom by -1 and including the subscript r for the repeated triaxial case:

$$M_r = \frac{\sigma_{1r}^{\,2} + \sigma_{1r}\sigma_{3r} - 2\sigma_{3r}^{\,2}}{\sigma_{1r}\varepsilon_{1r} + \sigma_{3r}(\varepsilon_{1r} - 2\varepsilon_{3r})} \qquad \text{Eqn.3-24}$$

Where	ε_{1r}; ε_{3r}	Repeated principal strains,	$\varepsilon_{1r}=\varepsilon_{1(2)}-\varepsilon_{1(1)}$; $\varepsilon_{3r}= \varepsilon_{3(2)}-\varepsilon_{3(1)}$
	σ_{1r}; σ_{3r}	Repeated principal stresses,	$\sigma_{1r}=\sigma_{1(2)}-\sigma_{1(1)}$; $\sigma_{3r}=\sigma_{3(2)}-\sigma_{3(1)}$

Then by substituting this equation into the second equation of Equation 3-18:

$$v = \frac{\sigma_1[\sigma_1\varepsilon_1 + \sigma_3(\varepsilon_1 - 2\varepsilon_3)] - (\sigma_1^{\,2} + \sigma_1\sigma_3 - 2\sigma_3^{\,2})\varepsilon_1}{2\sigma_1\sigma_3\varepsilon_1 + 2\sigma_3^{\,2}(\varepsilon_1 - 2\varepsilon_3)} \qquad \text{Eqn.3-25}$$

By dividing the top and bottom by $-2\sigma_3$ and including the subscript r for the repeated triaxial case the equation becomes:

$$v = \frac{\sigma_{1r}\varepsilon_{3r} - \sigma_{3r}\varepsilon_{1r}}{2\sigma_{3r}\varepsilon_{3r} - \varepsilon_{1r}(\sigma_{1r} + \sigma_{3r})} \qquad \text{Eqn.3-26}$$

Equations 3-27 and 3-28, below, are then found from 3-24 and 3-26 respectively, by substitution with Equations 3-5 and 3-7 for the case when u=0. If u≠0 then Equation 3-17 and those following equations would need restating with σ' in place of σ and p' in place of p.

These parameters can be defined in terms of p-q space and volumetric and shear strain (by applying the equations defined above), again for the triaxial situation by:

$$M_r = \frac{9p_r q_r}{9p_r \varepsilon_{sr} + q_r \varepsilon_{vr}} \quad \text{Eqn.3-27}$$

$$v = \frac{4.5p_r 3\varepsilon_{sr} - q_r 2\varepsilon_{vr}}{9p_r \varepsilon_{sr} + q_r \varepsilon_{vr}} \quad \text{Eqn.3-28}$$

Where:
- p_r Repeated mean normal stress, (p_2-p_1)
- q_r Repeated deviator stress, (q_2-q_1)
- ε_{vr} Change in volumetric strain, $(\varepsilon_{v2}-\varepsilon_{v1})$
- ε_{sr} Change in shear strain, $(\varepsilon_{s2}-\varepsilon_{s1})$

3.5 SUMMARY

Pavement materials exhibit two distinct types of behaviour when placed under the traffic loads:

- Elastic behaviour, which determines the load spreading ability of the layer, which manifests as cracking due to fatigue of the upper layers, and;
- Permanent deformation, which causes a build up of irrecoverable deformation, which after a number of load applications becomes apparent as rutting.

The structural capacity of a road pavement is often determined by the most critical structural behaviour (as above) in one of the layers that make up the pavement.

Although analytical mechanistic pavement design procedures are recognised as being superior to the traditional empirical methods there are some difficulties in establishing the parameters required for this type of design. Analytical mechanistic design requires fundamental material properties such as values for resilient modulus and Poisson's ratio. Work correlating empirical and analytical methods have shown considerable errors resulting in the conclusion that a direct test method of measuring the material properties accurately is required – repeated load triaxial is one such method. In general the stresses experienced by an element of soil or aggregate in a pavement structure under traffic loading can be simulated in the laboratory on representative samples using the repeated load triaxial apparatus.

The principle of repeated load triaxial tests is to simulate an element in a pavement by manufacturing a specimen of road construction material and applying similar load conditions to those that might be experienced in the field while measuring the deformation experienced by the specimen.

The purpose of the repeated load triaxial test is to determine the elastic parameters of materials, namely the resilient strain parameters and permanent strain parameters. Due to the stress dependent behaviour of these materials the elastic parameters must be determined at a number of stress levels.

Road construction specifications require all materials to be tested for suitability. In order to use road construction materials optimally a better understanding of the properties of these materials under traffic loading is necessary. This may be possible using sophisticated test apparatus to characterise the materials under simulated traffic loading, however, these sophisticated tests are often prohibitively expensive.

The loading applied to a specimen under repeated load triaxial testing is simplified by ignoring the rotational stresses applied when a load passes over a element. This limitation is made up for in the relative simplicity of the repeated triaxial apparatus in comparison to other test methods however. Thus this work only considers repeated load triaxial testing of materials in order to understand the behaviour of materials in a laboratory.

There are basically three different repeated load triaxial apparatus configurations as follows:

- Conventional repeated load triaxial test, with constant confining pressure;
- Internal vacuum repeated load triaxial test, with constant confining pressure;
- Variable confining pressure repeated load triaxial test.

Possible errors in testing material due to apparatus produced factors are as follows:

- Specimen Alignment
- Seating Errors and Bedding Errors
- End Friction or Restraint
- System Compliance
- Deformation Measurement

It is often desirable to depict the successive states of stress that exist in material as the specimen is loaded and unloaded. These stress points have co-ordinates namely mean normal stress and deviator stress and if these points are connected with a line or a curve called a stress path.

Stress paths, plotted in p-q space, allow different stress regimes to be applied to a test specimen in a repeated load triaxial apparatus. Theoretically it is possible to apply both comprehensive and tensile stresses to a specimen however unbound materials will only allow small tensile stresses due to pore water suctions and inter-granular friction.

Since unbound granular materials are used in layers nearer the surface of the pavement structure it may be argued that it is more important to correctly categorise these materials, rather than subgrade soils, since they are more heavily stressed.

In order to characterise unbound materials there are two important material parameters, namely resilient modulus and Poisson's ratio. Both are ratios describing the strain response to stress. Although often treated as properties of a material, it will be shown in Chapter 5 that strain has a non-linear relationship with applied stress for typical pavement foundation materials, thus these parameters are non-constant, and depend on the stress level for their magnitude.

4 FACTORS THAT INFLUENCE THE BEHAVIOUR OF MATERIALS IN PAVEMENTS

4.1 INTRODUCTION

Much effort has been devoted to characterising the resilient behaviour of road construction materials. To accommodate the non-linearity of these materials the resilient response is usually defined by resilient modulus and Poisson's ratio or alternatively, the use of shear and bulk moduli has been suggested. This will be discussed in the forthcoming chapters.

Rut development is a common mode of failure in flexible road pavements. The prediction of the permanent deformation development within a pavement structure is extremely complex, and it is probably due to this that, in comparison to resilient behaviour, less research has been devoted to the permanent deformation development in road construction materials.

For the design of pavements using road construction materials it is important to consider how the resilient and permanent behaviour varies with changes in other influencing factors as discussed below.

Although stress has a major effect on granular material behaviour, there are other characteristics that are likely to affect the behaviour of granular material under loading.

4.2 ENVIRONMENTAL CONDITIONS (MOISTURE IN PAVEMENTS)

The structural design of road pavements is usually based on material properties in the soaked condition (highly saturated). It is possible to examine the effect of various moisture conditions on pavement performance, particularly equilibrium conditions and seasonal fluctuations, provided the appropriate material properties can be measured. It is not usually realistic, however, to design pavements on the equilibrium condition, because temporarily abnormally wet conditions may have an overriding effect due to the greatly weakened properties of the pavement material layers at that time.

Where roads are constructed on clayey subgrades, unbound granular subbases are often placed directly on the subgrade or in the case of weak foundations on a low quality capping layer comprising local material that may contain some plastic fines. This construction practice can lead to water travelling upward through the high capillary rise subgrade into capping layer and subbase. The general principles of the interaction between unbound materials and water in the foundation and upper granular layers will be discussed here.

The foundation must perform in the longer term as a support system to the completed pavement. The stress levels applied will be much lower since the traffic loading will largely be supported by the pavement structure and the expected future moisture condition should have been anticipated during the pavement structure design. It is probable that, some time after construction, the equilibrium water condition will be reached. If this is higher than the moisture content during construction, failure could occur due to weakening of the foundation. The wetting of unbound granular base and subbase layers can cause a substantial reduction in material strength and stiffness, depending on the composition of the material.

4.2.1 Principles of Unbound Material and Water Interaction

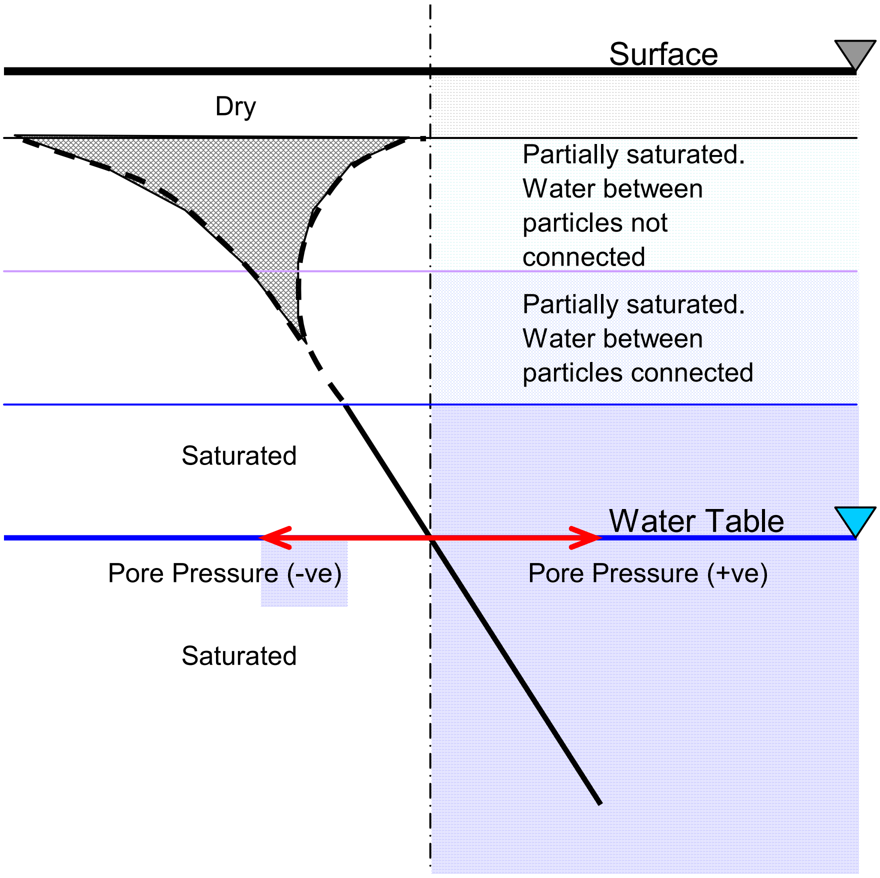
Soils and granular materials may be defined as an assemblage of discrete particles with variable amounts of water and air among the particles. Solid particles are in contact with one another forming the soil skeleton and the spaces between them form a system of interconnecting voids or pores.

There are three states in which these materials may exist with respect to water and air content:

i)	Dry	Voids contain only air
ii)	Saturated	Voids are full of water
iii)	Partially saturated	Voids contain both water and air

This is shown schematically in Figure 4-1.

Figure 4-1 Pore Pressure in Pavements



When loaded the compressive forces are considered to be positive. It is easy to envisage that, in a dry soil, the compressive forces are transmitted between particles at their points of contact. Therefore, the stiffness is directly related to the density of the material.

In the case of saturated materials, all of the voids in the material are filled with water. This may occur during periods of severe flooding and it can have severe consequences. Since the material and the water are virtually incompressible the application of a compressive stress will lead to a substantial build up of pore water pressure particularly in poorly drained conditions. Saturated granular materials also develop excess pore-water pressure under repeated loading. As pore-water pressure develops the effective stress in the material decreases with a subsequent decrease in both strength and stiffness of the material. Therefore, considering positive pore water pressure occurring in a fully saturated soil, then the effective stress as defined by Terzaghi (1943) states that:

$$\sigma' = \sigma - u \qquad \text{Eqn.4-1}$$

Where: σ' Effective stress
σ Total stress
u Pore water pressure

This principle was developed for saturated conditions and is therefore only valid in case of positive pore water pressures ($u > 0$). In case of partial saturation, negative pore water pressures ($u < 0$) occur and this law cannot be used directly.

The partially saturated condition in soils and unbound granular materials is far less simple. Under these conditions, which are the norm for roads, capillary forces retain the water in the unbound material layer against the gravitational forces that endeavour to drain the water out of the material and against evaporation of the water through the surface or side slopes. These capillary forces are caused by the curved air-water interfaces in the voids of the material. The tensile stress with which water is held in the material is termed the suction and since it increases the effective stress in Equation 4-1 it is a positive quantity.

However, if the water is connected in a continuous column and no upward flow is assumed, although in reality due to evaporations upward flow is expected, then the pore water pressure is hydrostatic. If the water particles in the soil matrix are discontinuous then the suction in each of the particles of water will take on the tension as established by the surface tension. In certain cases this may result in huge values, although the overall effect on effective stress will be less marked as the suction only acts at those points where particles of water exist and not uniformly across the whole soil. Therefore based on the complexity of the partially saturated conductions this part of the curve may vary from high to low tension as shown in Figure 4-1 by the hatched region.

4.2.2 Suction

Croney (1977) stated that water is held in the soil matrix by absorption and surface tension forces at a pressure less than atmospheric, which he called soil moisture suction, or suction. He defined suction as a measure of the pressure required to abstract water from a sample of soil, which is free from external loading. Suction is expressed on the pF scale, which is the common logarithm of the height in centimetres of an equivalent column of water.

Different materials types have different soil-suction characteristics. Coarse granular materials have small surface forces because of the low surface areas and the nature of the surfaces and, therefore, capillary forces dominate. Consequently, the suction potential is zero when these materials are saturated and completely dry. In both cases, no curved air-water interfaces exist and, therefore, no capillary forces are present. The maximum suction potential is reached somewhere in the unsaturated region of partly filled pores.

Clay particles have a far larger surface area per unit weight than larger granular particles. Furthermore, the surface of clay particles has an electric charge. Due to these electric charges the surface forces of clays may dominate over the capillary forces and can cause very large suction potentials in quite dry clays.

It is the effective stress that dominates aspects of soil behaviour such as compression and strength. Since both the total stress and the pore water pressure can be measured or calculated, depending on the situation in the laboratory or the field, the effective stress can easily be calculated. It is for this reason that most triaxial testing of clays is still being done under fully saturated conditions, thereby allowing Equation 4.1 to be used. Sands can be tested either in the fully saturated condition or completely dry. In the latter case, the whole problem of pore water pressures (positive or negative) simply does not occur. It was noted by Barksdale et al (1990) that the determination of unsaturated soil strength is particularly problematic since the accurate determination of the pore air pressure within the material is very difficult indeed.

4.2.3 Material Stiffness Related to Water Content

In addition to its effect on soil strength, suction also has a significant effect on the stiffness of soils. Since this stiffness plays an important role in pavement design and analysis, the influence of suction should be taken into account.

As stated above the standard approach used in pavement design, for the determination of stiffness parameters of unsaturated soil is to test the specimens that are prepared at the worst possible field moisture content and analysing the test results in terms of total stresses. Thompson and Robnett (1979), for instance, tested unsaturated subgrade soils for resilient characteristics using a repeated load triaxial apparatus. The specimens were prepared at and above the optimum moisture content and the resilient modulus was calculated as the ratio of the applied deviator stress to the recoverable axial strain of the triaxial specimen. The degree of saturation of the soils was shown to have a considerable effect on the resilient modulus, but no attempt was made to relate the resilient modulus to suction.

The degree of saturation of most untreated granular materials has been found to affect the elastic stiffness of road construction materials in both laboratory and in situ conditions. It is generally agreed that the resilient characteristics of dry and partially saturated granular materials is similar, but as complete saturation is approached the resilient behaviour may be affected significantly {Haynes and Yoder (1963), Hicks and Monismith (1971), Smith and Nair (1973); Barksdale and Itani (1989), Dawson et al (1996); Vuong (1992); Heydinger et al (1996)}

In resiliency testing of soils at high degrees of saturation, care should be taken when dealing with low permeability soils. Volume change can readily turn the negative pore water pressure (suction) in the triaxial specimen into a positive pore water pressure, since the low permeability prevents sufficient drainage. This may lead to a drastic reduction of the resilient modulus or even to specimen failure. In granular materials with a high permeability, these problems of positive pore water pressures do not occur provided the triaxial specimen is allowed to drain freely. Because of the relatively low value of suction in these materials, test results can be interpreted in terms of total stresses in the case of testing for practical purposes.

For research purposes, test results should be interpreted in terms of effective stresses. Providing that the material does not contain a significant amount of clay, this can be performed by testing the material in the completely dry condition. As suction will be caused by capillary effects, when completely dry this will be equal to zero. Effective stresses are then equal to total stresses and test results can be interpreted correctly without the need for measurement of suction.

For subgrade soils that contain significant amounts of clay, the simple approach of testing in the completely dry condition cannot be used. The clay causes a large suction in the material, which is at a maximum in the dry condition, and can therefore not be ignored. The resilient modulus of saturated clayey materials is usually expressed as a function of two stress variables, namely the confining stress and the deviator stress. Fredlund et al (1975) stated that, in case of unsaturated soils, a third stress variable (the suction) should be added.

Later, from experiments, Fredlund et al (1977) showed that the influence of the confining stress was negligible compared to the effect of the deviator stress and the suction and suggested the following equation:

$$\log M_r = c_{1d} - m_{1d} q \qquad \text{Eqn.4-2}$$

Where c_{1d} and m_{1d} are functions of soil suction

It must be noted that expressing the resilient modulus as a function of these three stress variables requires the measurement of the pore air pressure and pore water pressure during the repeated load triaxial test and Fredlund et al (1977) reported serious technical problems with this measurement.

It is also reported that the preparation of triaxial specimens at a specific suction level is cumbersome. It took Paute et al (1986) a minimum of one month per specimen to reach a specified suction level. The triaxial specimen was placed between two high air-entry porous discs, which were connected to a water vessel placed at a preset level below the triaxial specimen, thereby obtaining the required negative pore water pressure. By connecting a geotextile, which was placed between the triaxial specimen and the membrane down the side of the specimen, to the atmosphere, the pore air pressure was kept at atmospheric pressure.

An analysis of the results presented by Brown et al (1975) in terms of suction show a linear relationship between resilient modulus and suction at constant deviator stress. A similar relationship is reported by Finn et al (1972) determined with data from tests on a road subgrade. Dehlen (1969), working with silty clay, showed a measured suction decreasing with increasing depth below a road surface and a corresponding linear decrease in resilient modulus with suction. Croney (1977) states that soil moisture suction is an overriding factor in determining the elastic modulus for saturated clays. Fredlund et al (1975) measured the resilient modulus and suction of a till and a clay and found that the modulus increased with increasing suction, but at a decreasing rate.

In pavements the total time actually under load is small {Knight and Blight (1965)} and it is therefore reasonable to assume that there is some drainage although individual load pulses themselves are undrained. France and Sangrey (1977) allowed some continuous drainage during their cyclic tests, but not sufficient to inhibit the build up of pore pressures. Brown et al (1977), Anderson et al (1976), and Overy (1982) have all considered this problem. The general conclusion is that samples on the wet side of critical, with positive excess pore pressures, expel water and become stiffer while those on the dry side, with negative residual pore pressures will tend to imbibe water and soften

It was argued by Lekarp et al (2000a) that it is not the degree of saturation that influences the material behaviour but rather that the pore pressure response controls deformational behaviour. Mitry (1964), Seed et al (1967), and Hicks (1970) and Pappin (1979) all reported that if the test results are analysed on the basis of effective stresses, the resilient modulus remains approximately unchanged at any particular effective stress.

Saturation of unbound granular materials also affects the resilient Poisson's ratio. Hicks (1970) and Hicks and Monismith (1971) reported that Poisson's ratio is reduced as the degree of saturation increases.

As stated above excessive pore pressure reduces the effective stress, resulting in diminishing permanent deformation resistance of the material {Haynes and Yoder (1963); Barksdale (1972a); Maree (1982); Thom and Brown (1987); Dawson et al (1996)}. The stress-strain behaviour of soils and granular materials can be improved significantly by draining the system {Lekarp et al (2000a)}.

4.3 COMPACTION (DENSITY) OF PAVEMENT LAYERS

During road construction, engineers strive for high material compaction, which increases the density of the material, and is well known to alter its response to static loading, causing it to become both stiffer and stronger. Heavy compaction equipment is used to densify relatively thin layers of subgrade, subbase, base, and surfacing. After constructing a particular layer, this layer becomes a temporary working surface for the next layer and so on. Usually these compaction stresses are the greatest stresses to which a particular layer is ever subjected.

The application of large vertical stresses during construction is reported to cause lateral stresses to develop that become locked into both granular bases and cohesive subgrades {Sowers, et al., (1957); Uzan, (1985); Selig, (1987); Duncan and Seed, (1986)}. Thom and Brown (1988) and Brown and Selig (1987) stated that the effect of density on resilient modulus is relatively insignificant but that density appeared to have some influence on Poisson's ratio {Barksdale and Itani (1989)}.

It is surmised that the reason that density does not have a great effect on resilient modulus is because when a load is applied to a pavement material the particles move relative to one another changing the stress between them accordingly (σ' and u). As the load is removed they return to their original positions and thus this is independent of density, density may, however, effect the distance moved. Poisson's ratio is affected by the variation in density and this thought to be because a more dense material is more likely to transmit movement through particles, therefore vertical load will cause more horizontal movement for a more dense material.

The effect of density, as described by degree of compaction, has been regarded in previous studies as being significantly important for the long-term behaviour of granular materials {Holubec (1969); Barksdale (1972b), Barksdale (1991); Allen (1973); Marek (1977); Thom and Brown (1988)}. Resistance to permanent deformation in these materials under repetitive loading appears to be highly improved as a result of increased density.

4.3.1 Compaction of Granular Bases

It was shown by Selig (1987) that in a granular layer, large plastic lateral strain develops in the bottom of the layer during the first cycle of loading. Upon subsequent loading cycles, however, the response rapidly approaches an elastic condition. The lateral stress in the bottom of the granular layer, in both the loaded and unloaded condition, gradually increases up to about 50 load cycles. After 50 load cycles the lateral stresses in both the loaded and unloaded states were found to be in the order of 20 times greater than before the first load cycle. The horizontal stress in the unloaded condition, however, was larger than the stress existing when fully loaded which was not true when loading first started. These important findings indicate that the stress increment in the bottom of the granular base caused by loading becomes tensile after a relatively few load cycles. However, when added to the existing residual lateral stress, a net compressive state of stress exists in the bottom of the granular layer. It was concluded by Selig (1987) that the residual lateral stress is the most important factor limiting permanent deformation of the bottom of the granular base and therefore is also an important factor in determining the appropriate stress state at which to evaluate the resilient modulus. The residual lateral stress is relatively large and can be expressed by:

$$\sigma_{hr} = K_0 \sigma_0 \qquad \text{Eqn.4-3}$$

Where:	σ_{hr}	Residual lateral stress
	σ_0	Vertical overburden stress
	K_o	Coefficient of lateral earth pressure

The coefficient of lateral earth pressure for this condition is greater than unity but less than the passive coefficient of earth pressure that represents a limiting condition of failure due to lateral movement outward. For a granular material the passive coefficient of earth pressure is equal to:

$$K_p = tan^2\left(45 + \frac{\theta}{2}\right) \qquad \text{Eqn.4-4}$$

Where: φ Angle of internal friction for the granular material

The work of both Selig (1987) and Uzan (1985) indicate the great importance in properly considering the residual stress that exists in a granular base in the analyses used in mechanistic based pavement design procedures.

4.3.2 Compaction of Cohesive Subgrade

Uzan (1985) stated that residual lateral pressures were observed for both cohesionless and cohesive soils. Uzan (1985) and Duncan and Seed (1986) proposed methods of analysis for predicting residual lateral stresses due to compaction of cohesive soils. Sowers, et al (1957) investigated the residual lateral stresses produced by compacting soil in a steel mould. This study indicated that the residual stresses became less with increasing moisture content, and decrease sharply at moisture contents close to the optimum value. Also, Sowers, et al (1957) reported that, for the clay tested, the residual stresses induced by static compaction were higher than those induced by impact compaction. Relatively high values of residual stresses were measured due to compaction of soils in a confined cylinder, which does not duplicate the free field conditions existing during fill placement.

4.4 THE EFFECT OF STRESS LEVELS

The literature shows that stress level has a significant impact on resilient properties of road construction materials. However deviator stress has been found to be much less influential on material stiffness than confining pressure and the sum of principal stresses {Mitry (1964), Monismith et al (1967), Hicks (1970), Smith and Nair (1973), Uzan (1985), and Sweere (1990)}. Monismith et al (1967) reported an increase as great as 500% in resilient modulus for a change in confining pressure from 20 to

200 kPa and Smith and Nair (1973) observed that an increase of about 50% in resilient modulus was observed when the sum of principal stresses increased from 70 to 140 kPa.

Hicks (1970), Brown and Hyde (1975), and Kolisoja (1997) have all reported that Poisson's ratio of unbound granular materials increases with increasing deviator stress and decreasing confining pressure, implying that the resilient Poisson's ratio is also influenced by the state of applied stresses.

It is well known {Lashine et al (1971) and Brown and Hyde (1975)} that there is a threshold value of repeated deviator stress magnitude, above which the sample eventually fails and below which an equilibrium state is reached regardless of the number of further cycles. Lashine et al (1971) reported that during triaxial testing the measured permanent axial strain settled down to a constant value that is directly related to the ratio of deviator stress to confining pressure.

4.5 LOAD DURATION AND FREQUENCY

Boyce et al (1976) showed that the limestone material they tested in a triaxial apparatus was subjected to stress history effects. They stated that pre-loading with a few cycles of the current loading regime and avoiding high stress ratios in tests for resilient response could reduce these. Brown and Hyde (1975) and Mayhew (1983) reported that resilient characteristics of unbound granular materials are basically insensitive to stress history, provided the applied stresses are kept low enough to prevent substantial permanent deformation in the material. Therefore, large numbers of resilient tests can be carried out sequentially on the same specimen, to determine the resilient parameters of the material.

The general view regarding the impact of load duration and frequency on the resilient behaviour of granular materials is that these parameters are of little or no significance {Seed et al (1967), Morgan (1966), Hicks (1970), Boyce et al (1976), and Thom and Brown (1987)}

Permanent strain continuously increases under repeated loading {Morgan (1966); Barksdale (1972); Sweere (1990)}. Morgan (1966), for instance, applied up to

2,000,000 load cycles and reported that permanent strain was still increasing at the end of the tests. Barksdale (1972) concluded that permanent axial strain in untreated granular materials accumulates linearly with the logarithm of the number of load cycles. However, Brown and Hyde (1975) noted that an equilibrium state was established after approximately 1,000 load applications. Paute et al (1986) stated that the rate of increase of permanent strain in granular materials, under repeated loading, decreases constantly to such an extent that it is possible to define a limit value for the accumulation of permanent strain.

4.6 LOADING HISTORY

Brown and Hyde (1975) showed that resilient characteristics of unbound granular materials are not affected by loading history. They showed that a large number of stress paths can be applied to a specimen for the determination of resilient parameters, provided that the stresses applied are kept well below the failure line for the material.

Brown and Hyde (1975) reported that permanent strains in unbound granular materials are, on the contrary, affected significantly by loading history. Therefore, several triaxial specimens have to be tested to obtain the relationship between stress ratio applied and permanent deformation. Each test usually involves a large number of load applications on each specimen, which renders the determination of permanent strain characteristics to be quite time consuming. This is probably the reason why far less data is available on permanent deformation of unbound granular materials than there is on resilient deformation.

Kalcheff and Hicks (1973) investigated the effects of load duration and load frequency on the resilient modulus and showed no quantifiable effect of load duration on the resilient modulus.

4.7 THE EFFECT OF MATERIAL PROPERTIES

Thom and Brown (1988), Brown and Selig (1991) and Raad et al (1992) all concluded that the stiffness of road construction material is, in some degree, dependent on particle size and its distribution.

Selig and Roner (1987) stated that flaky particles, at stresses which do not lie along the failure line, increase the shear strength of granular material, however, the disadvantage being that an increase in the flakiness may cause problems of breakage, abrasion, increased permanent strain and decreasing stiffness of the material.

Aggregate type may have a significant effect on the resilient modulus when other parameters such as grading, density and applied stress are kept constant. Barksdale and Itani (1989) tested different granular materials with the same grading and they found that angular materials had a higher resilient modulus than rounded gravel, the increase being about 50% at low mean normal stress conditions decreasing to about 25% at high mean normal stress levels.

Thom (1988) and Thom and Brown (1989) carried out repeated load triaxial tests on different granular materials, and concluded that the resilient modulus of granular materials at low strain levels may be influenced by particle texture, that a correlation exists between elastic stiffness and the surface friction properties of materials, and that high resilient modulus and good load spreading properties in the pavement may be expected from material with angular to sub-angular shaped particles and a very rough surface when compared with material with sub-rounded or rounded particles and a smooth surface.

4.8 SUMMARY

The basic principles of road design, the dependence on traffic loading prediction and assessment of the subgrade strength have remained unchanged for centuries. Naturally, it is fundamental that the properties and characteristics of the construction materials are known and understood.

Not only are the properties of the materials themselves important but it is also important that an understanding of the way that different material layers in a pavement mechanism interact. For example cracking of an asphalt surface layer is probably a consequence of weaknesses in the granular base. Pavement behaviour can also be influenced by other imposed conditions such as environmental conditions, moisture, loading magnitude, duration and frequency, for example.

For the design of pavements using road construction materials it is important to consider how the resilient and permanent behaviour varies with changes in other influencing factors. A number of factors are likely to influence the behaviour of unbound materials in pavements, namely:

- Environmental conditions (moisture in pavements), particularly suction which has a significant effect on the stiffness of soils.
- Compaction (density) of pavement layers.
- The effect of stress levels.
- Load duration and frequency.
- Loading history.
- The effect of material properties.

It is concluded that the preparation of triaxial specimens at specified suction levels requires sophisticated equipment and, more importantly, a great deal of time. A more practical approach is the preparation of triaxial specimens at field moisture content and measuring the suction in the specimen using some device.

All road construction materials, unbound granular materials and subgrade soils, are heavily stress dependent.

5 ANALYSIS OF THE BEHAVIOUR OF THE MATERIALS BY MODELLING

5.1 INTRODUCTION

The main objective in testing the materials in the laboratory is to determine the material parameters, such as the resilient modulus, from the test results. These parameters can be used to define the suitability or otherwise of the material in road construction. This may apply to foundation materials that will inevitably form part of the pavement structure or imported borrow materials to be used to construct the upper structural layers. A number of constitutive relationships have been developed and are presented here. These relationships require that model coefficients are determined for each of the materials tested during the 'Science Project'. By incorporating the parameters obtained from the relationships into mechanistic pavement design the most economic use of available materials in terms of layer thickness and structural life can be determined. It is important that accurate coefficients for the relationships are obtained and the magnitude of any errors appreciated by practising engineers.

During this work (as will be presented) recorded values, stresses and strains, are obtained from repeated load triaxial testing. This chapter describes how these results may be analysed using constitutive relationships in which model coefficients and subsequent material parameters can be determined. The aim of these models is to predict the behaviour of the materials under traffic loading.

5.2 MODELLING THE EXPERIMENTAL DATA

To analyse different pavement structures, constructed using different materials, certain material characteristics must be determined. These vary from simple values determined from simple laboratory testing, for example failure characteristics yielding relationships between resilient modulus and CBR, to more complex characteristics which, in practice, are often estimated from simple material characteristics rather than the results of complex of testing.

In order to examine the effect of using different constitutive relationships for the analysis of road construction materials a range of different relationships have been considered. These relationships vary from relatively simple models (k-theta model) to more complex models (Mayhew model) described later. Some of the more simple models are used in practice, for example in the AASHTO (1993b) and Austroads (1992) procedures. With the advent of more powerful personal computer based analytical computer methods, such as finite element analysis, more complex models will become more common in pavement design. Of course, it does not necessarily follow that the greater the complexity of the relationship the better the accuracy of the predictions made by the model. Naturally these relationships require accurate input data, since if estimations of complex material characteristics are made from inappropriate material tests then the models will make erroneous predictions.

During the analysis of an artificial specimen in the forthcoming chapter it will be shown that some consideration of the applied stress is necessary in order to compare results from different sources since the applied stresses are not always identical. The computation of bulk and shear modulus for this artificial material was calculated and from these values the resilient modulus and Poisson's ratio were determined allowing comparisons to be made. During the repeated load triaxial laboratory tests on materials strains are measured under predetermined stress conditions and since these materials are heavily stress dependent it is important that both stresses and strains are considered during modelling.

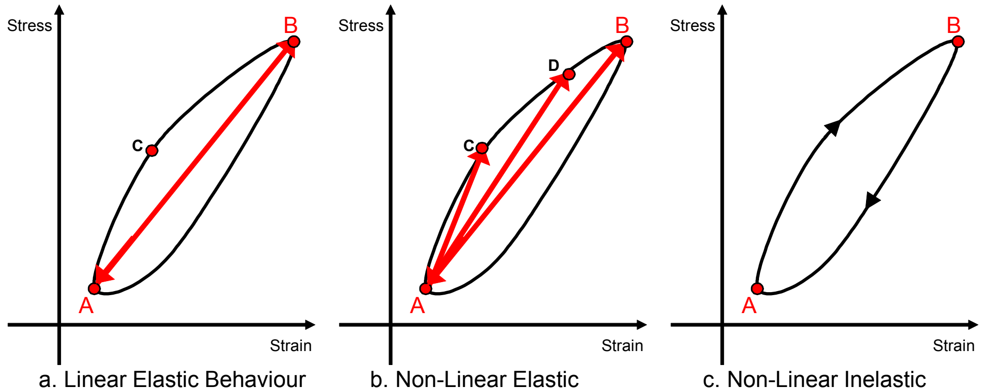
Suppose that the observed strain response to a cyclic application of a deviator stress is represented by the black path in Figure 5-1, ACBA (which would be a typical response). Then this is normally modelled in one of three ways, namely:

- Linear elastic behaviour — The stress strain curve is assumed to be a straight line connecting A to B under loading and to return along the identical path (B to A) when unloaded (Figure 5-1a). Thus a single value for resilient modulus can be found, namely the slope of the vector, irrespective of the magnitude of B, even if the actual stress applied (and resulting strain) is indicated by (for example) point C.

- Non-linear elastic behaviour — The stress strain curve is assumed to be the lines that connects A to C, D and B. This curve is assumed to apply under both loading and unloading. It is common to describe this behaviour by the equivalent linear modulus which would represent the same maximum stress-strain. Thus the resilient modulus varies with the magnitude of maximum stress applied, and may be defined by the slope of the vector which connects the relevant point (e.g. the lines A-C, A-D or A-B in Figure 5-1b).

- Non-linear inelastic behaviour — The stress strain curve follows the exact path as shown by the black line in Figure 5-1c for both the loading element and the unloading element of the curve. If this behaviour is to be described by an equivalent linear modulus (because only the end points are of interest) then it is modelled no differently from the preceding case. However, if the strain path is to be computed then some other model will be required - for example a tangent resilient modulus which varies with the magnitude of stress and varies for the loading and unloading cases.

Figure 5-1 Definition of Linearity and Elasticity



a. Linear Elastic Behaviour b. Non-Linear Elastic Behaviour c. Non-Linear Inelastic Behaviour

Note: the black loop it taken to be the actually path as the material is loaded and unloaded.

The non-linearity of these characteristics of a road construction material is illustrated in Figure 5-2, this figure shows the resilient modulus and Poisson's ratio as a function of the deviator stress applied on a triaxial specimen. The resilient modulus is clearly dependent on the deviator stress, whereas the Poisson's ratio proves to be more constant for this example for the specimen of London Clay. For the sample of Soft Limestone shown in Figure 5-3 the reverse can be seen, the Poison's ratio is dependent on the deviator stress, whereas the resilient modulus appears more constant.

Figure 5-2 Stress Dependency of the Resilient Modulus and Poisson's Ratio for a Sample of London Clay

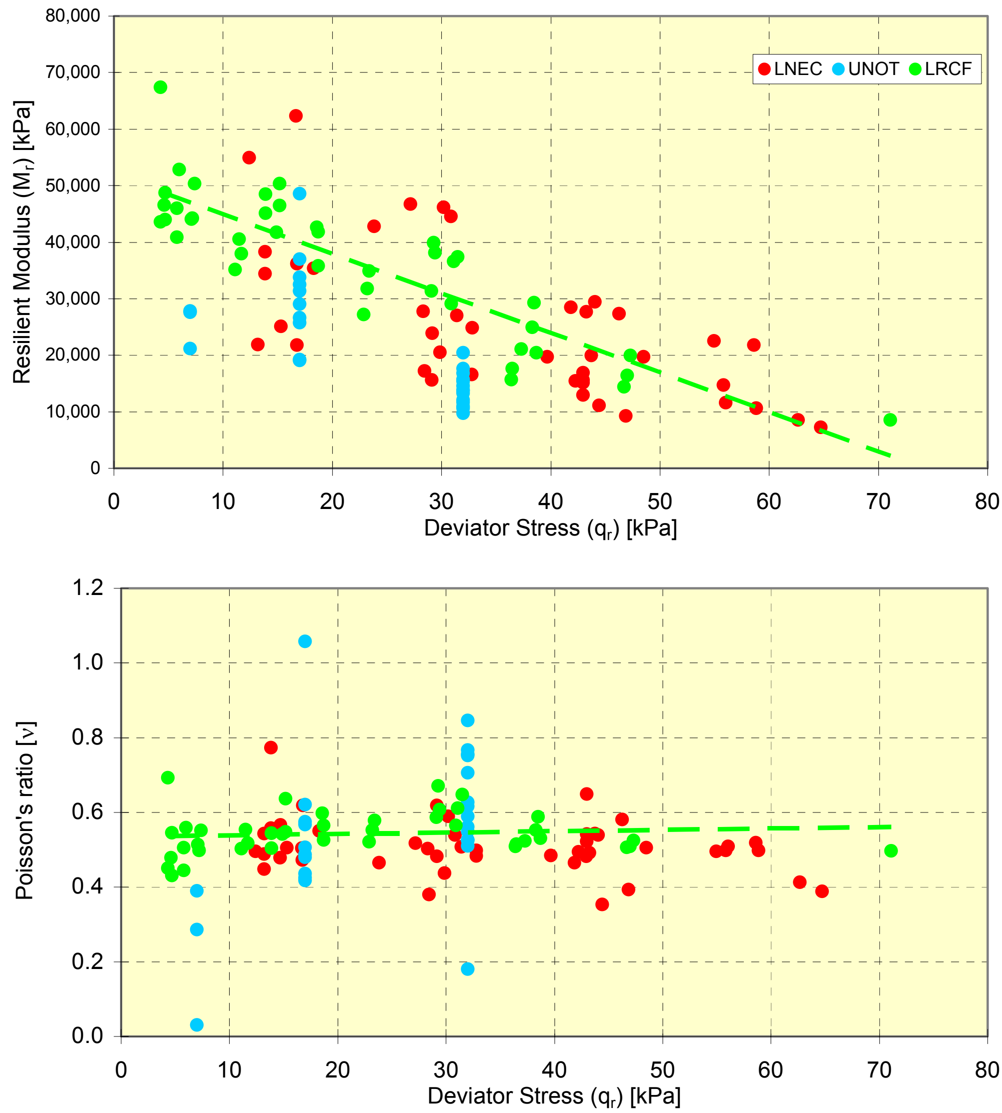
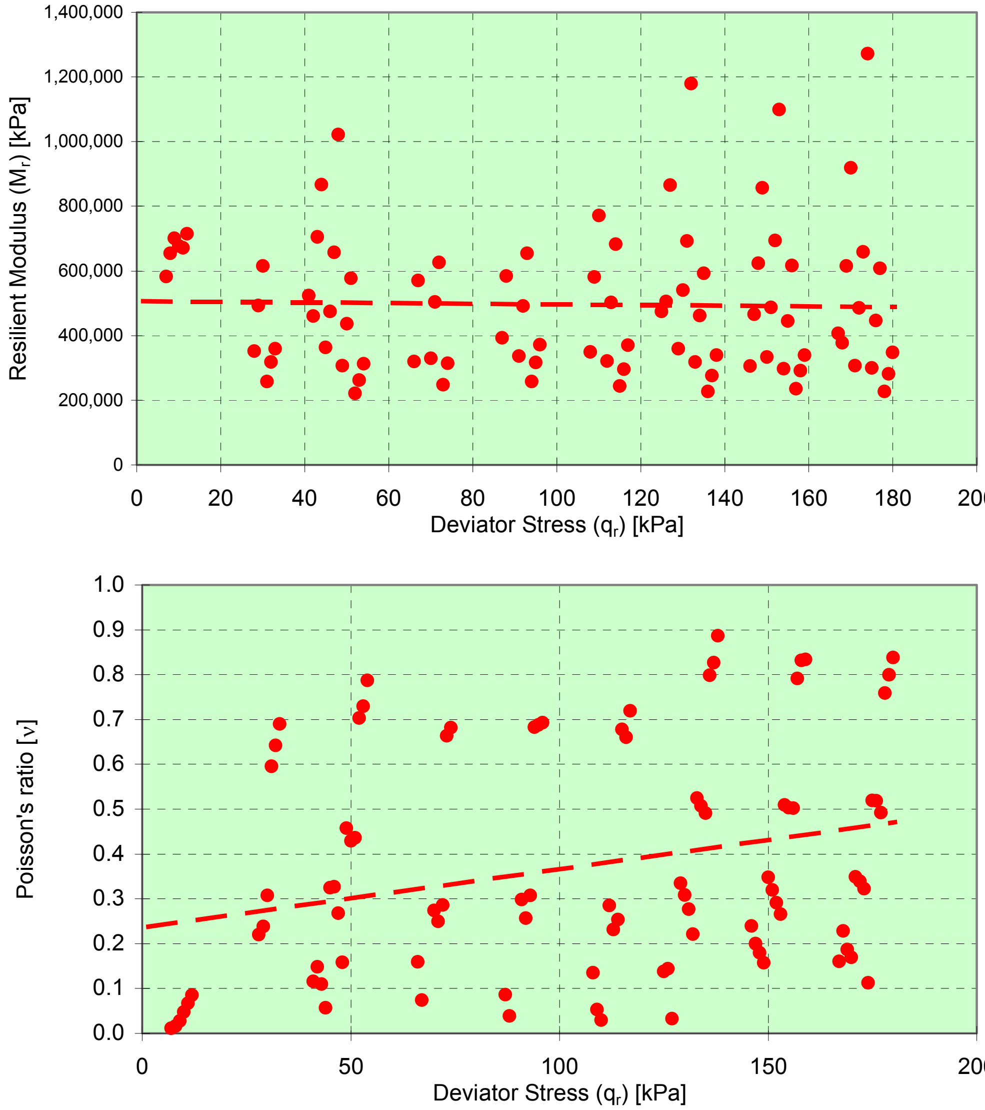


Figure 5-3 Stress Dependency of the Resilient Modulus and Poisson's Ratio for a Sample of Soft Limestone



Empirical pavement design methods require the resilient modulus and Poisson's ratio to be constant {Ahlborn (1963)}, therefore, it is desirable to define the most relevant resilient modulus and Poisson's ratio for each material specimen tested. Since these parameters are stress dependent, they should be defined at the characteristic stress level pertinent to the material in the pavement. In Chapter 2 the characteristic stress levels, in terms of deviator and mean normal stresses, were defined for an unbound granular material and a subgrade soil.

5.3 CONSTITUTIVE RELATIONSHIPS TO DEFINE THE BEHAVIOUR OF MATERIALS

The models and corresponding parameters that are to be used for the analysis can be divided into three categories, namely those:

- For all road construction materials;
- For subgrade soils, and;
- For unbound granular materials.

In order to allow the dimensions of the coefficients used by these models to be non-dimensional a constant p_a has been introduced, p_a is taken to be atmospheric pressure. This non-dimensionality is not generally the case in the original forms of the models.

5.3.1 Models for all Road Construction Materials

Simple Linear Elastic Model

The simple linear elastic model simply defines a characteristic resilient modulus and Poisson's ratio. These parameters are calculated from the experimental data for each stress path. They are calculated using the equations in Chapter 3 and are thus stress dependent since they depend on the stress applied to the specimen.

$$M_r = Constant$$
$$v_c = Constant$$
Eqn.5-1

Where M_r Characteristic resilient modulus, MPa
ν_c Characteristic Poisson's ratio, constant

The k-theta model

The k-theta model {Hicks and Monismith (1971)} is an early model and in quite common use. It is a simple model for the resilient modulus that has 2 model coefficients (k_1 and k_2) and two material parameters (M_r and ν).

$$M_r = k_1\left(\frac{3p_2}{p_a}\right)^{k_2} \quad or \quad M_r = k_1\left(\theta\right)^{k_2}$$

Eqn.5-2

$$v_c = Constant$$

Where:

p_2	Maximum mean normal stress, ($3p_2$ = theta), kPa
p_a	Atmospheric pressure, p_a = 100 kPa
k_1, k_2	Model coefficients

The k-theta model has been used for design of new pavements {Thompson (1992)} or pavement evaluation {Brown and Almeida (1993)}. However, it has some drawbacks, the Poisson's ratio is not modelled and a constant characteristic value for this parameter needs to be defined. Although experimental values show that Poisson's ratio is not constant, for this work, the Poisson's ratio is defined by the value determined at the characteristic stresses mentioned above. Secondly, this model does not allow directly for any change in the deviator stress applied, which means it is best used for low shear levels which is not generally the case for pavements, particularly the upper layers {Uzan (1985)}. Thirdly the model has been developed from simple laboratory triaxial test results in which the initial deviatoric stress is always zero and in which the confining pressure is constant.

The second drawback was investigated by Shackel (1973), May and Witczak (1981) and Uzan (1985) and they modified the model in order to include the deviatoric stress. Uzan et al (1992) proposed a similar expression to the k-theta model but solving the dimensional problems, they also showed that the model for Poisson's ratio is also able to predict values larger than 0.5.

<u>The Uzan Model</u>

An improvement on the k-theta model, for all road construction materials, is the Uzan model {Uzan et al (1992)}, which included the deviatoric stress and has 3 model coefficients. Again the Poisson's ratio is not modelled and a characteristic value needs to be chosen.

$$M_r = k_3 \left(\frac{p_2}{p_a} \right)^{k_4} \left(\frac{q_2}{p_a} \right)^{k_5}$$

$$v_c = Constant$$

Eqn.5-3

Where: q_2 Maximum deviator stress, kPa
k_3 , k_4, k_5 Model coefficients

5.3.2 For Fine Grained Subgrade Soils used in Road Construction

<u>The Brown Model</u>

The Brown model as reported by Hyde (1974) and modified by Gomes Correia (1985) to include material suction at a specific moisture content, with two model coefficients can be applied to fine grained subgrade soils that often comprise the road foundation. Once again the Poisson's ratio is not modelled and a characteristic value needs to be defined.

$$M_r = A \left(\frac{s}{q_2} \right)^{B}$$

$$v_c = Constant$$

Eqn.5-4

Where s Suction, kPa
A, B Model coefficients

The Loach model

The Loach model {Loach (1987)} also includes material suction as a function of moisture content, with 2 model coefficients can also be applied to fine grained subgrade soils that comprise the road foundation. Again the Poisson's ratio is not modelled and a characteristic value is defined.

$$M_r = C\left(\frac{s}{q_2}\right)^D \frac{q_2}{p_a}$$

$$v_c = Constant \qquad \text{Eqn.5-5}$$

Where C, D Model coefficients, or constants

5.3.3 For Unbound Granular Materials used in Road Construction

It was reported by Karaşahin (1993) that because neither resilient modulus nor the Poisson's ratio is constant for unbound granular materials under loading making the assumption that they are constant may cause serious problems in predicting the behaviour of these materials since they show non-linear stress-dependent behaviour.

Originally Domaschuk and Wade (1969) used bulk modulus (K) and the shear modulus (G) rather than Young's modulus and Poisson's ratio in order to explain stress-dependent behaviour of sand. This approach was taken up by Pappin (1979) and Pappin and Brown (1980) who divided the measured strains into volumetric and shear instead of axial and radial strains using K and G. They developed a non-linear resilient behaviour model, called "contour model" which can directly be applied to non-linear numerical analysis methods. The model is based on the repeated load triaxial test results and they concluded that the shear strain is path-dependent although the volumetric strain is not.

Boyce (1980) developed a non-linear isotropic model with G and K using the theorem of reciprocity (i.e. there is no net loss of strain energy) also expressing it in the volumetric and shear parts.

The Boyce model

Boyce (1980) developed a non-linear elastic stress strain relationship for aggregates based on laboratory testing and mechanistic theory. This model originally had 3 parameters but was modified during the 'Science Project' {Galjaard et al (1993)} to include a fourth parameter (p*, which is defined graphically in Figure 5-4), which is an indirect measure of apparent cohesion in the material due to suction and interlock effects, and for the general case where there is a change of stress from the start to the end of the stress path is as follows:

$$\varepsilon_{vr} = \Delta\left\{p_a^{1-n} \cdot p^n \cdot \frac{1}{K_a}\left[1 - \left(\frac{K_a \cdot (1-n)}{6\,G_a}\right)\left(\frac{q}{p+p^*}\right)^2\right]\right\}$$

Eqn.5-6

$$\varepsilon_{sr} = \Delta\left\{p_a^{1-n} \cdot p^n \cdot \frac{1}{3\,G_a} \cdot \frac{q}{p+p^*}\right\}$$

However, since the deviator stress, q, has an initial value in a triaxial laboratory test, approximately equal to zero (i.e. $q_1 = 0$), the equation becomes:

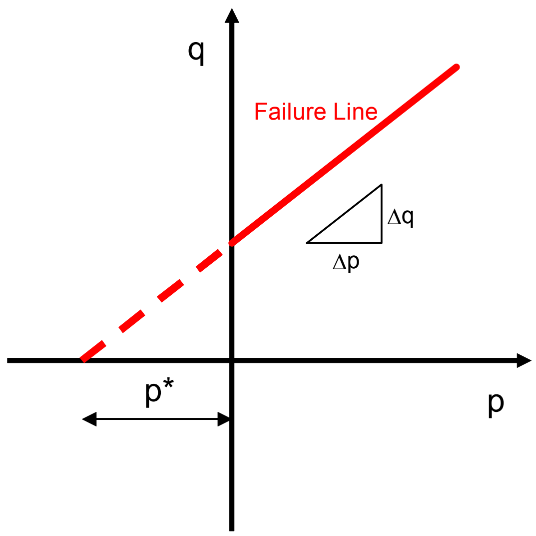
$$\varepsilon_{vr} = p_a^{1-n} \cdot \frac{1}{K_a}\left[p_2^n - p_1^n - \left(\frac{p_2^n \cdot K_a \cdot (1-n)}{6\,G_a}\right)\left(\frac{q_2}{p_2+p^*}\right)^2\right]$$

Eqn.5-7

$$\varepsilon_{sr} = p_a^{1-n} \cdot p_2^n \cdot \frac{1}{3\,G_a} \cdot \frac{q_2}{p_2+p^*}$$

Where:	G_a, K_a	Model coefficients that relate to the shear and bulk modulus of the material
	n, p*	Material coefficients

Figure 5-4 Determination of the p* Coefficient



The Mayhew model

When the Boyce equations are analysed in a non-linear parameter evaluation model the values for the parameters G_a and n for volumetric and shear strain are different for each. This creates the problem of having two values for the same parameter for the same model for the same data. Although this difficulty can be overcome by weighting each relationship, it illustrates the difficulty in fitting the measured material behaviour (and probably the genuine material behaviour) to the models. This is overcome in the Mayhew model (see below) by having five parameters instead of the three in the Boyce model. Again p* was introduced during the 'Science Project'' and thus here.

The Mayhew model {Mayhew, (1983)} with 6 parameters is thus:

$$\varepsilon_{vr} = \Delta\left\{p_a^{1-m} \cdot p^m \cdot \frac{1}{K_a}\left[1 - \beta\left(\frac{q}{p + p^*}\right)^2\right]\right\}$$

Eqn.5-8

$$\varepsilon_{sr} = \Delta\left\{p_a^{1-n} \cdot p^n \cdot \frac{1}{3G_a} \cdot \frac{q}{p + p^*}\right\}$$

The shear strain equation is identical to the Boyce model. Again, since the initial deviator stress (q_1) is approximately equal to zero, the equation becomes:

$$\varepsilon_{vr} = p_a^{1-m} \cdot \frac{1}{K_a}\left[p_2^m - p_1^m - p_2^m \cdot \beta\left(\frac{q_2}{p_2 + p^*}\right)^2\right]$$

Eqn.5-9

$$\varepsilon_{sr} = p_a^{1-n} \cdot p_2^n \cdot \frac{1}{3G_a} \cdot \frac{q_2}{p_2 + p^*}$$

Where: β, m Model coefficients

It was noted by Allaart (1989) that these models (Boyce and Mayhew) model the volumetric strain poorly whereas they predict the shear strain quite well.

5.4 SUMMARY

A number of constitutive relationships have been developed in which model coefficients and subsequent material parameters can be determined. The aim of these models is to predict the behaviour of the materials under traffic loading. These vary from simple values determined from simple laboratory testing, for example failure characteristics yielding relationships between resilient modulus and CBR, to more complex characteristics which, in practice, are often estimated by simple material characteristics rather than the results of complex of testing.

The models and corresponding parameters that are to be used for the analysis can be divided into three categories, namely those:

- For all road construction materials;
- For subgrade soils, and;
- For unbound granular materials.

It is not the intension of this work to develop a model that predicts the behaviour of road construction materials under traffic loading. As with the pavement design methodology where limiting strains are correlated to permissible traffic loading the work of others is used to determine the possible accuracy of laboratory testing and pavement design using these tools. The main reasons for selecting the models described above are:

- That a range of models has been selected, from simple to complex models, some are commonly used for commercial pavement design and other are only used in research, and;
- The limitations of the software used to analyse the data, certain models are integrated into the FENLAP {Almeida (1991)} software.

6 TRIAXIAL TEST APPARATUS

6.1 INTRODUCTION

There exist many varieties of triaxial apparatus used for the repeated load testing of pavement construction materials. These apparatus vary in size and sophistication. These variations often depend on the characteristics of the material being tested, for example an apparatus designed to test full sized crushed rock (granular base) will be much larger than an apparatus used to test fine-grained subgrade soils. The sophistication of apparatus generally varies with budgets, with some apparatus using expensive sophisticated different loading systems, deformation measuring devices and data capture mechanisms, while others use more simple mechanical devices.

6.2 COMMON METHODS OF MEASURING STRAIN ON SPECIMENS IN THE LABORATORY

Spring loaded clamps around the specimen, studs and vanes placed within the specimen and blocks or targets glued on the membrane have all been used as reference points between which deformations are measured on the specimen. On samples of rock or heavily stabilized materials wire resistance strain gauges are sometimes glued directly on the specimen.

The most common method of measuring specimen deformation is by electronic measurement devices such as Linear Variable Differential Transformers (LVDT) or Proximity Transducers (PT). LVDTs have an energising coil wound coaxially with a receiving coil between which flux is transferred in proportion to the position of a metal armature that slides along the axis of the coils. The coils are relatively bulky, but the armature is very thin (diameter about 2 mm) and lightweight. The arrangements for the support of the LVDTs vary. In general, however, in the case of small softer specimens a frame supports the weight of the LVDTs whereas the larger, stronger, specimens are better able to support the weight of the LVDTs. The PTs record the movement of the specimen without being physically connected to it, therefore an external frame always supports PTs. PTs measure the change in inductance of a coil as a ‘target’ piece of foil (fixed to the specimen) is brought into the flux field around the transducer thus altering its inductance. The response of the PTs is highly non-linear, but a signal conditioner linearises this over a certain specified range of position of the target from the sensor.

Strictly, these instruments measure the displacement of a specimen under loading; but as these displacements are used to compute strain (axial and radial) the latter term will be used here.

6.2.1 Spring-Loaded Rings

The use of spring-loaded rings placed around the specimen to measure axial displacement is frequently used. The rings are usually constructed from either aluminium or Plexiglas consisting of two pieces that are hinged on one side and have a spring-loaded connection on the other. A LVDT (or PT) is placed between the ring openings and the specimen displacement is measured. Two rings are generally used in order that axial deformation can be measured between the two rings by either two or three LVDTs between them. The rings are placed at either the $^1/_4$ or $^1/_3$ points in from each end of the specimen as discussed earlier. Dehlen (1969) reported using these rings so that the ring touched the specimen along two short segments. Hicks (1970) and Barksdale (1972b) also used spring-loaded rings to measure radial deformation of triaxial specimens.

Tilting of the rings will influence the accuracy of axial strain measurements. This is probably a result of either barrelling of the specimen or misalignment, accurate alignment is essential in order that tilting does not occur {Chisolm and Townsend (1976)}.

It was observed during permanent strain measurements using spring-loaded rings that the measured strains exhibited a greater scatter after 50,000 load cycles than before in relation to strain measurements measured in some other less accurate way {Barksdale (1972b)}. The proposed hypothesis was that the rings underwent small amounts of slip over an extended number of load applications. Boyce and Brown (1976) and Pezo et al (1991) also suggest that ring slippage could be a potential problem in measuring resilient modulus.

To prevent slip between the clamps and the membrane, Chisolm and Townsend (1976) simply placed a small quantity of epoxy glue on top of each of the clamp contact points with the rubber membrane. Sweere (1990) used individual LVDT support blocks glued directly to the membrane. Static tests by Miller, as reported by Burland and Symes (1982), indicate that relative slip between the specimen and the enclosing membrane does not occur until near or after failure. The work of Miller indicates that pasting lightweight clamps or blocks to the membrane should be satisfactory for the stress levels normally employed in resilient modulus testing.

6.2.2 Studs and Pins

To eliminate the possibility of slip between the radial measuring apparatus and the specimen, several researchers have placed studs or pins in the specimen {Boyce and Brown (1976) and Paute et al (1986)}. These studs are embedded within the granular specimen during specimen manufacture. A second part of the stud is then attached to the embedded stud through the membrane. Boyce and Brown (1976) consider the metal studs, which protruded into a granular base material, as simply an artificial aggregate.

Small cross-shaped vanes have been pushed into a soft cohesive soil to which pins are attached as reported by Brown (1979). To minimize applying load on these pins, four LVDTs, which are supported externally, were used to measure the deformation within the specimen at two locations.

6.2.3 Non-Contacting Sensors

A number of different sensors are available, which do not contact each other including inductive, optical, ultrasonic, and pneumatic types. Ultrasonic type non-contacting sensors have a low sensitivity, while pneumatic type non-contacting sensors are large {Linton et al (1988)}. As a result, neither appears to be suitable for resilient modulus measurement. However, axial strain on triaxial specimens has been successfully measured using non-contacting measurement systems including both inductive proximity gauges and optical scanners. Proximity gauges have been used more frequently to measure radial deformation for evaluating Poisson's ratio and volume change, than for axial strain measurement.

Inductive Proximity Sensors

Inductive proximity sensors have been successfully used by Dupas et al (1988) to measure axial strain in gravels and clayey sands. However, for axial measurement although the proximity sensors themselves are non-contacting, blocks or pins must still be attached to the specimen, as is the case in LVDT based axial measurement systems. Thus, if relative displacement is to be measured, then the instrumentation must be supported by the pins or blocks and hence ultimately by the specimen. If, however, absolute displacement is used to measure displacement of a reference point, then only lightweight targets need to be attached to the specimen. During this work absolute radial measurements were conducted at mid-height of soil specimens using two opposing PTs fixed to the frame opposite metallic rectangles that were glued to the specimen at Nottingham and Lisbon.

Although proximity sensors are quite accurate, their use poses some perhaps minor problems associated with adjusting the sensors to within the correct range when employed for axial displacement measurement {Barksdale et al (1990)}. Also, they are moderately expensive.

Optical Scanners

Measurements of displacement during repeated load triaxial test have been performed using optical scanners, by attaching reflective targets on specimens {Moore et al (1970), Allen and Thompson (1974), and Knutson and Thompson (1978)}. The scanner, which is located outside the triaxial cell, optically monitors the movement of the targets.

The important advantage of using optical scanners over other systems is that only very light targets are attached to the specimen. If the displacement of each target is to be measured simultaneously, however, the same number of optical heads is required as the number of targets. Moore et al (1970) employed a square triaxial cell chamber to keep from distorting the light beams. A circular cell was used, however, in the later systems adopted by Allen and Thompson (1974) and Knutson and Thompson (1978) with the latter reporting no loss in accuracy. Moore reports a high resolution of the system, being able to make measurements to better than 0.001 mm over a 1.8 mm range. Optical scanner heads, unfortunately, are expensive and are probably not suitable for routine testing.

Pore Pressure Measurement

Most triaxial testing equipment measures pore pressures at the base of the sample, and because some end restraint will always be present, the measured pore pressure will not be representative. Some researchers, such as Sangrey et al (1969), used very low frequencies of loading to allow time for pore pressure equalisation. Others, such as Koutsoftas (1978), allowed time after faster cyclic loading for the pore pressure to equalise. In either case the end effects will still distort the recorded pore pressure but the second method has the advantage of testing at representative frequencies or rates of loading. It seems likely that the most accurate pore pressure measurements will be from a centre probe in the relatively uniform central section of the sample before pore pressure equalisation has taken place Hight (1983), assuming that the transducer itself does not cause any significant effects.

6.3 APPARATUS AND EQUIPMENT USED DURING THIS WORK

The following section contains a description of each of the participating laboratory's test apparatus and their corresponding instrumentation systems.

6.4 UNIVERSITY OF NOTTINGHAM

Two repeated load triaxial apparatus were used in this work at the School of Civil Engineering at the University of Nottingham, one to test subgrade soils and the other to test unbound granular base materials. These are shown in Photograph 6-1.

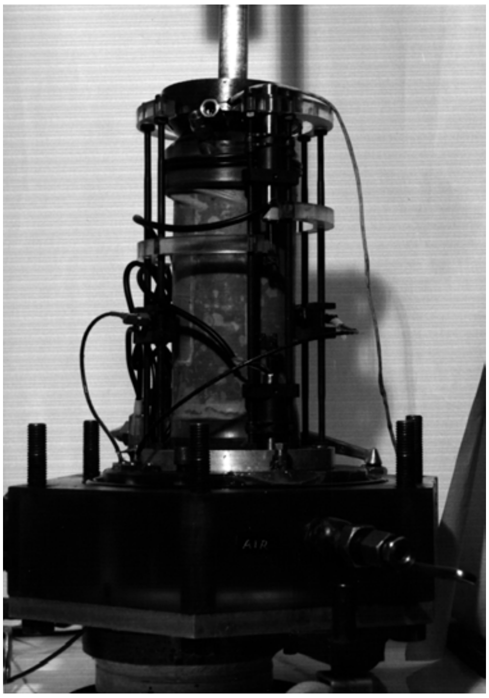
6.4.1 Variable Confining Pressure Apparatus (150 mm x 76 ∅mm) for Testing of Subgrade Soils

The Apparatus

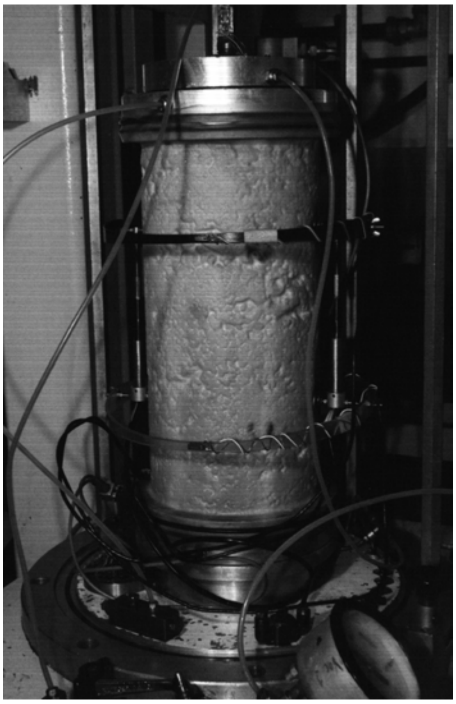
The servo controlled hydraulic triaxial testing facility was first developed in 1971 for testing fine grained soils and has since undergone a number of modifications {Loach (1987)}. The apparatus is contained in an air-conditioned laboratory and consists of a loading frame with two hydraulic actuators, one for axial load and the other for cell pressure. A pump supplies hydraulic power, at a normal operating pressure of

14 MPa. The control system permits the user to cycle both axial and confining pressure. The apparatus is illustrated in Figure 6-1.

Photograph 6-1 Apparatus at Nottingham



Variable Confining Pressure Apparatus (150 mm x 76 ∅mm) for Testing of Subgrade Soils



Variable Confining Pressure Apparatus (300 mm x 150 ∅mm) for Testing of Unbound Granular Materials

The axial load is applied to the specimen by connecting the load ram in the triaxial cell directly to the hydraulic actuator, thus tension can be applied axially. The axial loading system has a load capacity of approximately 12 kN, which allows a pressure of 2500 kPa to be applied on a specimen.

Silicone oil confining fluid is used in the triaxial cell (Dow Corning 200/20 cs) since it is non-conductive and therefore does not interfere with the electronic instruments. The confining pressure is applied by connecting the hydraulic actuator to a piston, which acts on the silicone oil to a maximum pressure of about 400 kPa. The feedback transducer is a strain gauged diaphragm pressure transducer, located in the cell. The servo control system compares the control signal with the feedback signals provided by the outputs of the axial load cell and the cell pressure transducers. The electronic system allows control of the cycling of both the confining stress and the deviator stress by limits of stress or by limits of deformation. The triaxial cell contains a specimen 150 mm high with a 76 mm diameter.

Specimen Manufacture

The fine-grained material was conditioned (by adding water or drying) and mixed to the required moisture content. The material was compacted into a steel mould of diameter 76 mm using a rammer weighing 4.54 kg (BS1377 Modified Proctor Hammer). The number of layers of material and the number of blows was determined by a method of trial and error until the required density at the specified moisture content was attained. Alternatively, some specimens have been manufactured by placing the mould on a vibratory table, placing the material in five layers in the mould and applying a surcharge to each layer for a fixed period of vibration. This method is described by Boyce and Brown (1976).

Instrumentation

An instrumentation support frame is placed around the specimen and the location of the axial and radial points of measurement marked on the specimen. Four axial locating cruciform vanes are pressed into the specimen at $^1/_3$ and $^2/_3$ of the specimen height and two metallic rectangles (25 x 35 mm) (aluminium foil) are fixed to the specimen by glue adhesion, at the mid-height, for the radial measurement. This is shown in Figure 6-1. Two 5 mm wide nylon gauze strips are placed vertically along the length of the specimen, to distribute the pore pressure between the top and the bottom of the specimen. A latex membrane is placed over the specimen and fixed by two rubber ‘O-rings’ to the top and bottom platens. A pin is screwed through the membrane into each locating vane and the LVDT armature connected to the pin. The specimen is then placed between upper and lower platens in the cell and the frame is fixed to the base of the cell. Four LVDTs (Figure 6-1) are connected to the frame and to each of the pins (cruciform vanes). The difference between the reading of deformation of the upper and lower LVDT in each pair allows axial strain to be computed. The radial deformation is measured at mid-height of the specimen using two opposing PTs fixed to the frame opposite the metallic rectangles.

The triaxial cell is sealed, placed in the loading frame and filled with fluid. The triaxial cells are fitted with castors to minimise the need to lift and carry the cells and limit specimen disturbance.

6.4.2 Variable Confining Pressure Apparatus (300 mm x 150 ∅mm) for Testing Unbound Granular Materials

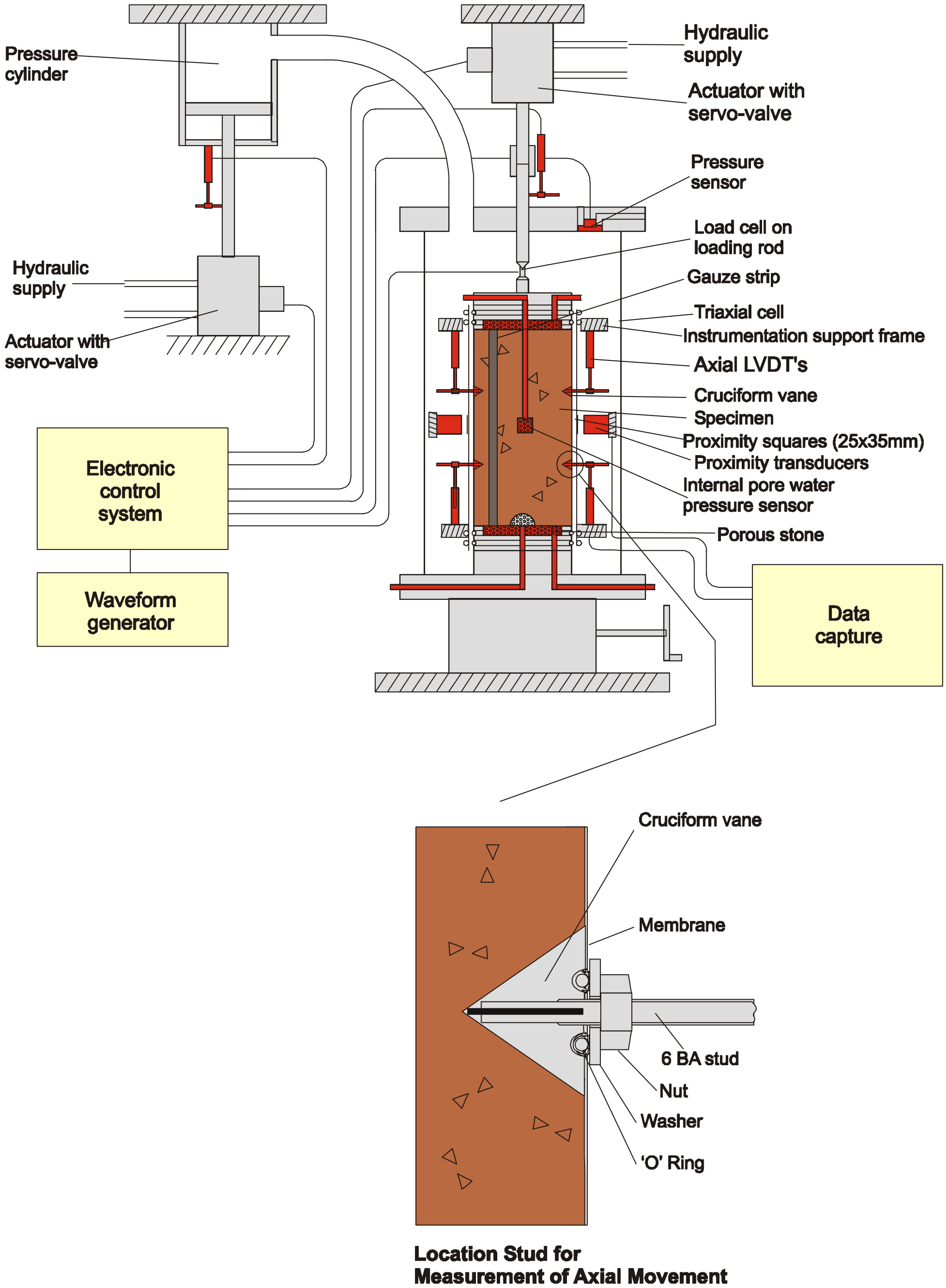
The Apparatus

This equipment was first developed at Nottingham in 1974 in order to provide a test facility with which to study the mechanical properties of unbound granular materials used in pavement construction. Facilities to cycle both the confining stress and the deviator stress were provided, to approximately represent the effects of repeated wheel loading in the pavement. The equipment allows for testing materials with a maximum grain size of 30 mm particle size.

The main components of the triaxial cell and servo-hydraulic loading systems for deviator and confining stresses are shown in Figure 6-2. The test specimen is housed in a sealed, pressurised triaxial cell. Silicone oil is once again used as the cell fluid.

Axial load is applied to the specimen by a hydraulic actuator and monitored by a load cell. Confining stress is applied through the silicone fluid surrounding the test specimen. A second hydraulic actuator loads a piston in a pressure cylinder that controls the fluid pressure. A pressure sensor in the triaxial cell monitors the pressure. The axial loading system has a load capacity of approximately 20 kN. This allows deviator stresses in the range 1200 kPa to be applied on 150 mm diameter specimens. A cell pressure of up to 400 kPa can be applied.

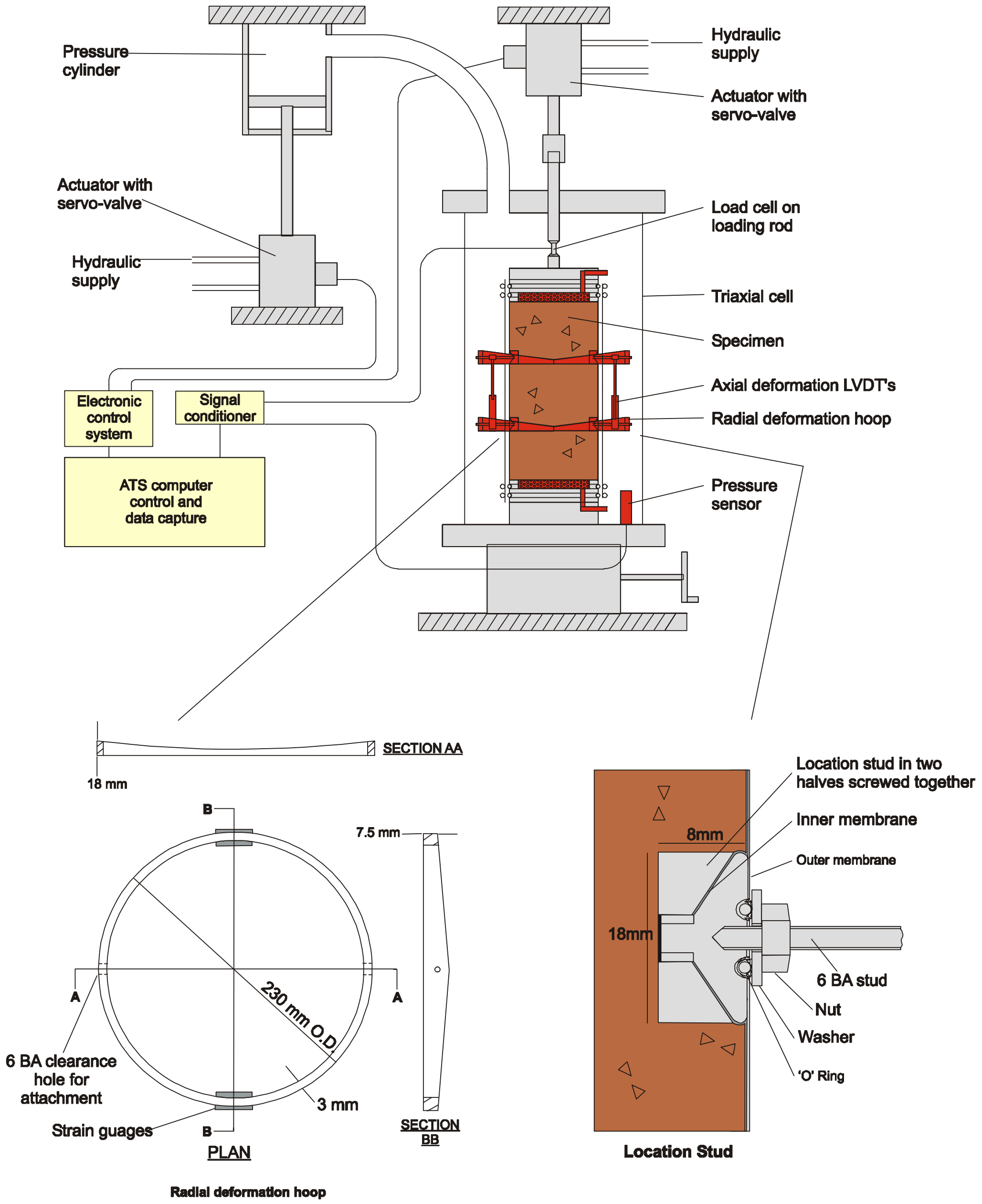
Figure 6-1 University of Nottingham - Variable Confining Pressure Apparatus (150 mm x 76 ∅mm)



Specimen Manufacture

The specimen is prepared in a four-piece aluminium split mould into which an inner latex membrane is held using a vacuum. Four "locating studs" are attached to the inner membrane. The test material is then placed in layers, each being subjected to a programme of vibration, while a small surcharge load is applied, thus enabling the density to be controlled. Once compaction is complete a top platen is placed on the specimen and the membrane is sealed to it using 'O-rings'. Then an internal vacuum is applied to the specimen thus allowing the mould to be dismantled and the specimen to be transferred to the cell base. A second outer latex membrane is placed on the specimen in case the first was punctured during the compaction.

Figure 6-2 University of Nottingham - Variable Confining Pressure (300 mm x 150∅ mm)



Instrumentation and Data Capture

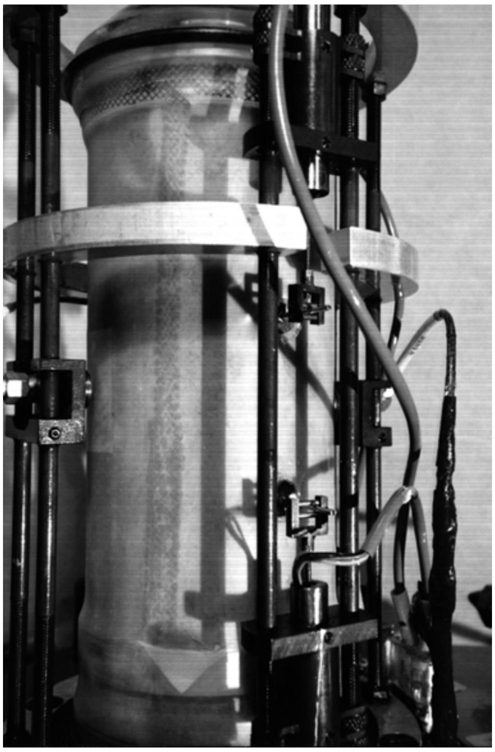
Four studs are used which allow two independent radial strain measurements to be made. The studs are typically placed over the central 150 mm of the specimen height, at ¼ and ¾ of the specimen height (to avoid any end effects) at 180° to one another as shown in Figure 6-2.

Two small LVDTs are attached between the studs (as above) to measure the axial movement during loading. Flexible strain-gauged rings are also attached to the locating studs, which measure the radial movement of the specimen under loading. These rings are made from casting epoxy (Araldite resin MY 778 resin HY 956) and weigh approximately 25 g.

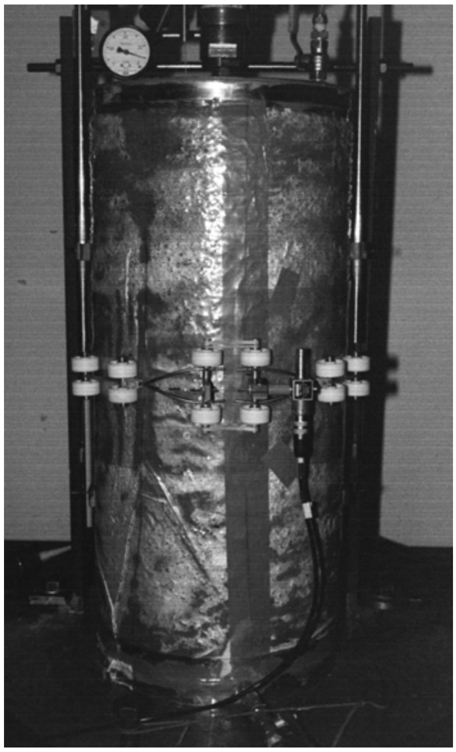
6.5 LABORATÓRIO NACIONAL DE ENGENHARIA CIVIL

The Laboratório Nacional de Engenharia Civil has two repeated load triaxial apparatus as shown in Photograph 6-2.

Photograph 6-2 Apparatus at Lisbon



Variable Confining Pressure Apparatus (150 mm x 76 ∅mm) for Testing of Subgrade Soils



Constant Confining Pressure Apparatus (600 mm x 300 ∅mm) for Testing Unbound Granular Materials

6.5.1 Variable Confining Pressure Apparatus (150 mm x 76 ∅mm) for Testing of Subgrade Soils

For the repeated load triaxial testing of soils the Laboratório Nacional de Engenharia Civil uses an apparatus very similar to that at the University of Nottingham, which was manufactured by the University. The loading characteristics and specimen manufacture are identical to those of the Nottingham apparatus. The computer control and data acquisition hardware used is a Hewlett Packard 3852A data acquisition unit and a Hewlett Packard 900 series 300 computer. The data acquisition software was written and is maintained by the Laboratory {Gillett (1994)}.

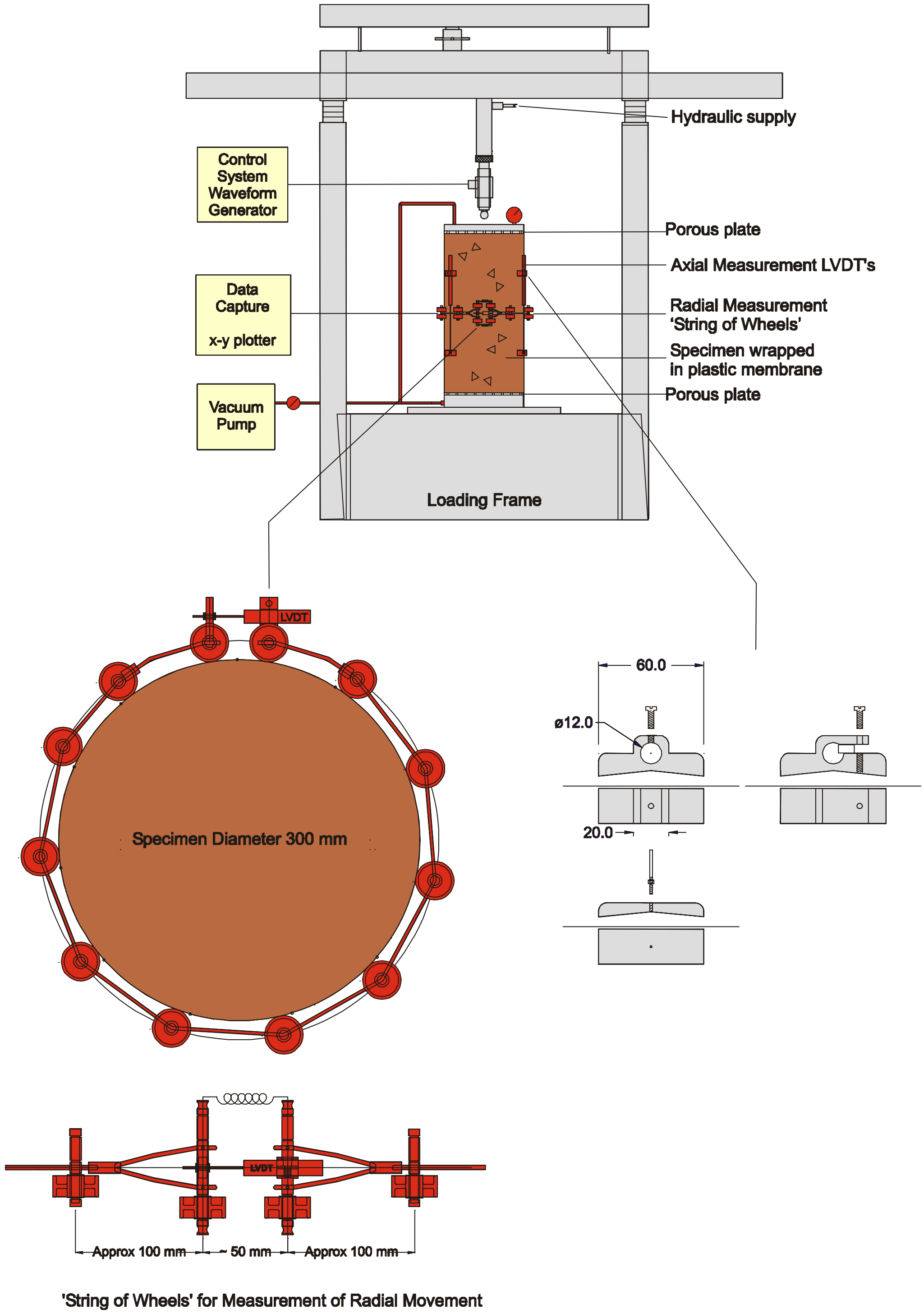
6.5.2 Constant Confining Pressure Apparatus (600 mm x 300 ∅mm) for Testing Unbound Granular Materials

This apparatus was developed by Nunes and Gomes Correia (1991), conceptually based on a triaxial cell used for testing rock-fills by Veiga Pinto (1983). It was developed for the testing of full sized single sized granular material used for railway ballast in Portugal. During this project it was modified to conduct repeated load tests on unbound granular materials of up to 40 mm grading. The triaxial specimen height is 600 mm with a diameter of 300 mm.

<u>The Apparatus</u>

The loading frame is constructed from standard mild steel sections of sufficient strength to withstand loading of up to 50 kN. The deviator stress is applied by means of a hydraulic jack attached to the loading frame, which applies a load to the top platen. Pressure to the jack is applied by means of an ENERPAC BVE-31 pump apparatus and an ENERPAC BIC-93 program control centre. This system can apply a maximum force of 25.7 kN, which imposes a maximum deviator stress on a specimen of 300 mm diameter of about 330 kPa. The maximum realistic operational frequency of loading, i.e. load-on load-off to load on again, of this system is about 0.5 to 0.6 Hz. The confining pressure is applied by means of a CENCO HYVAC7 vacuum pump that can apply a maximum vacuum of 70 kPa. The apparatus is shown schematically in Figure 6-3.

Figure 6-3 Laboratório Nacional de Engenharia Civil - Constant Confining Pressure Apparatus (600 mm x 300 ∅mm)



Specimen Manufacture

A rugged rubber membrane is placed inside a steel split mould of two halves and three sections high and the material is compacted in ten layers of approximately 10 kg (for materials having a specific gravity of around 2.65) and 60 mm height using a vibration hammer. In this manner the correct density is attained for specified moisture contents.

Once the specimen has been compacted and the split mould removed the rubber membrane is peeled off. This is done because of the possible confining pressure applied by the rubber membrane. The rubber membrane is replaced by a 0.3 mm thick oversized plastic membrane, which is made up of a plastic sheet wrapped loosely around the specimen and sealed with plastic tape and silicone sealant. Although this membrane is not very extensible, it is sufficiently oversize to allow the specimen to expand axially and laterally and does not apply a significant confining pressure to the specimen.

A vacuum is applied inside the membrane to simulate the confining pressure, thus there is no triaxial cell around the specimen as is usual with triaxial apparatus. Applying a sub-atmospheric pressure to the inside of the triaxial specimen simulates the confining stress. An advantage of not having a pressure cell is that the transducers for measuring the strains remain accessible during the test, thus small range differential transducers may be used (and manually adjusted should they go out of range).

Instrumentation and Data Capture

Axial strains are measured by means of an LVDT on the top platen and by two glue-on LVDT holders at ¼ and ¾ positions each on opposite sides of the specimen. The radial deformation is measured by means of a 'String of Wheels' wrapped around the specimen at mid-height, while a LVDT measures the increase or decrease of the circumference of the specimen and thus the radial deformation. The instrumentation is shown in more detail in Figure 6-3.

This system was based on a prototype utilised at The University of California at Berkeley. A steel cable is threaded through ten sets of wheels and attached to a LVDT holder at each end. A LVDT then measures the increase in the circumference of the specimen and thus the radial deformation. Each set of wheels comprises a pair of wheels on an axle through which a steel cable of diameter 2 mm, coated in plastic, is threaded. These axles are not fixed to the cable, thus the cable may move through the axles. The ends of the cable are attached to the LVDT holders. The ‘String of Wheels’ is then wrapped around the specimen and held together by two fairly stiff elastic bands, and the wheels are manually spaced around the specimen.

The method of data capture is by means of pen plotters. A load cell, manufactured by Automation Industries (TDC 205) between the loading jack and the top platen sends a signal to an XY plotter, and thus the load is recorded and monitored as it is varied manually, The LVDTs send signals via a wheatstone bridge to pen recorders. The vacuum is controlled by means of a bleed and is monitored manually at two pressure gauges one at the pump and the other through the top platen. This method of load control and data capture although functional is very time consuming and prone to operator errors.

6.6 LABORATOIRE REGIONAL DES PONTS ET CHAUSSÉES

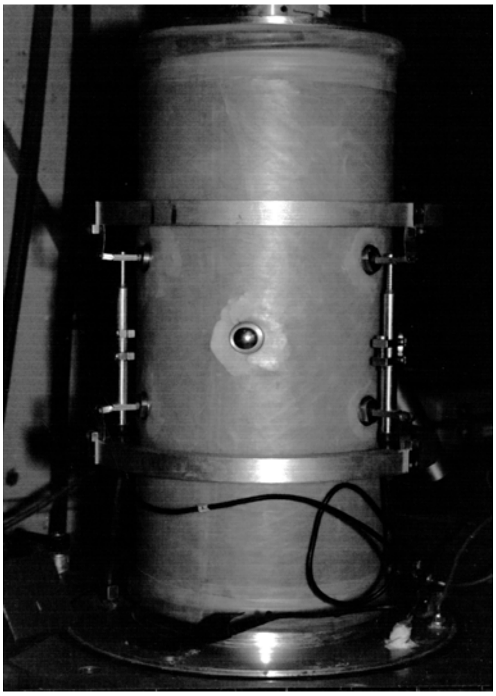
The Laboratoire Regional Des Ponts et Chaussées has two repeated load triaxial apparatus. One at Saint Brieuc for testing granular material, shown in Photograph 6-3, and the other for testing subgrade soils at Clermont Ferrand.

The Apparatus

This triaxial cell is based on the standard equipment manufactured by Wykeham Farrance with an adapted base to house 70 mm diameter specimens. The extra space between the specimen and the cell is used to mount the internal instruments in order to measure the deformations of the specimen.

In the cell top there is a Druck PDCR 22 transducer used to accurately monitor the cell pressure. The cell base has been modified to allow access for the cables of the measuring instruments, which are inside the cell. Both the axial load and the confining pressure systems are based on those developed at the University of Nottingham. The cell pressure is applied to the specimen by means of non-conducting silicon oil in the cell. This is schematically shown in Figure 6-4.

Photograph 6-3 Apparatus at Saint Brieuc



Variable Confining Pressure Apparatus
(320 mm x 160 ∅mm) for Testing
Unbound Granular Materials

6.6.2 Variable Confining Pressure Apparatus (150 mm x 70 ∅mm) for Testing of Subgrade Soils

Specimen Manufacture

The specimens are compacted in five layers, in a split mould, that is lined with a latex membrane. Each layer is compacted by a vibrating full faced surcharge to produce the required height and thus dry density. The anchors for the axial and radial measuring devices are attached to the mould and the material compacted around them. For dry sand, the material is compacted in a single layer on a vibrating table (frequency 50 Hz and amplitude of 0.42 mm) with a surcharge of 10 kPa. For moist soils, the moisture content is varied to achieve the correct density and, subsequently, the specimen is dried until the correct moisture content is attained. Some work was conducted on the uniformity of the specimen as a function of layers {Gomes Correia (1985)} and a specimen compacted in five layers was found to provide a uniform specimen when tested with a nuclear density method.

Instrumentation and Data Capture

The axial deformation is measured using four LVDTs diagonally opposite each other on the specimen, two in each plane at $^1/_3$ and $^2/_3$ the specimen height. Each LVDT is supported by a frame and attached to the specimen by means of a probe, anchored in the specimen, in a similar manner to the Nottingham cruciform vane. This is shown in Figure 6-4.

The radial deformation is measured by two LVDTs, supported by a frame, diagonally opposite each other in each plane at mid specimen height. On the end of the core of the LVDT is a flat disc (15 mm in diameter) which is held by a light spring to a dome that is attached to an anchor embedded in the specimen. This allows the radial deformation to be measured despite axial displacement of the dome.

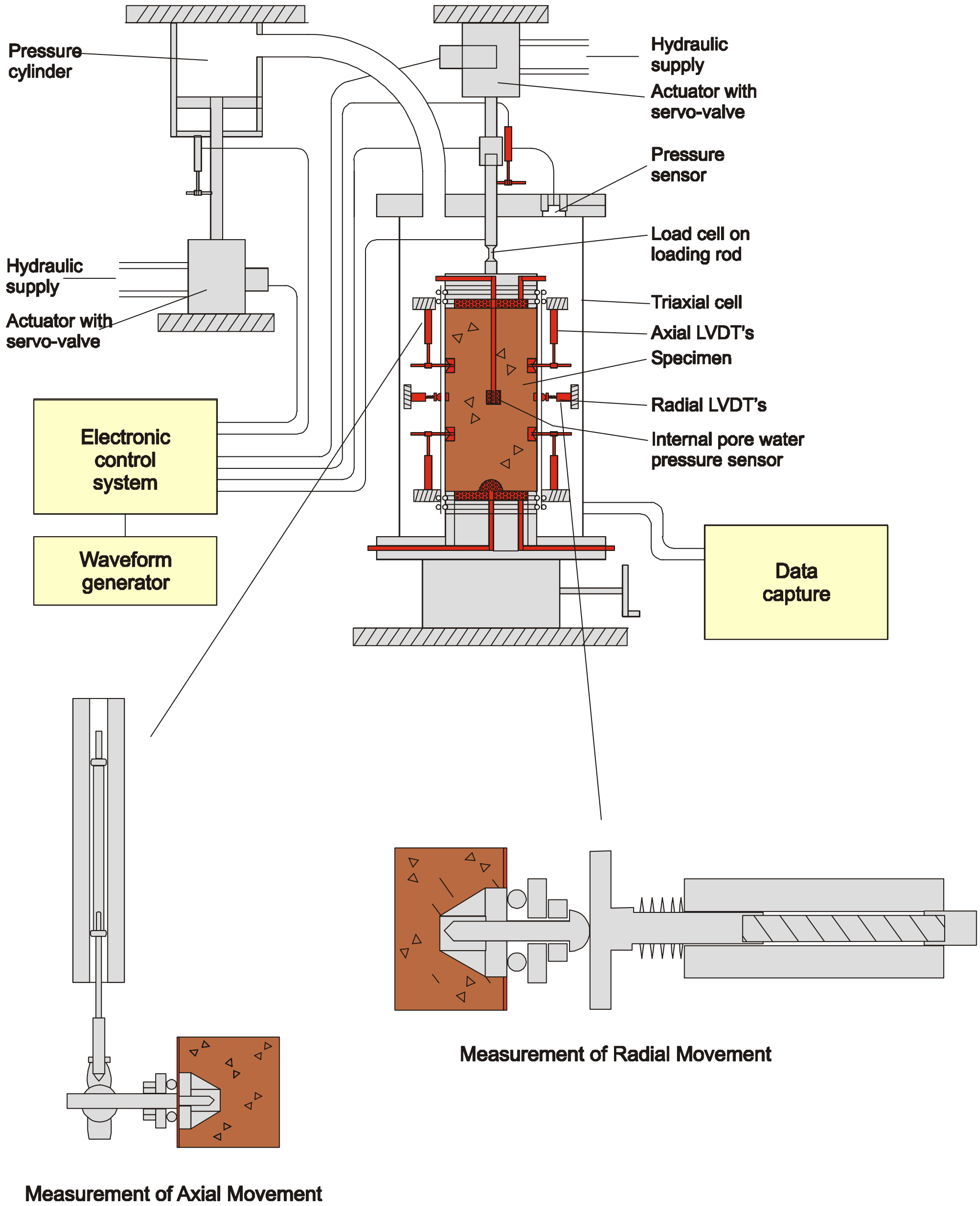
A Hewlett Packard series 200 computer (HP 9816S) processes the recorded data. All the equipment and software used for the data acquisition and processing was developed and is maintained by LRPC. The data acquisition system records the movement of the four axial transducers and four radial transducers. The number of measurements monitored by the data acquisition is limited to 50 cycles, under the maximum frequency of loading allowed by the apparatus.

The Apparatus

This repeated load triaxial apparatus was developed for the study of the behaviour of unbound granular materials. The specimen size is 320 mm high and 160 mm diameter. The major difference between this apparatus, and those used by other participating laboratories, is that the loading is pneumatically powered. The drainage is controlled at both ends of the specimen through porous plates. A load cell is positioned on the top platen thus frictional effects between the loading rod and cell are avoided and the cell pressure is measured by means of a pressure transducer in the cell. The maximum cell pressure is 500 kPa.

The maximum compressive force on the loading frame is 15 kN, which is 745 kPa on a specimen of diameter 160 mm. The pneumatic jack, applying the axial load, and the cell pressure cylinder are supplied by two different circuits, as illustrated by Figure 6-5. For each of them, two sensitive pressure regulators give the maximum and the minimum value of the pressure of the cycle. On each circuit, an electro-pneumatic distributor connects the line of the cell (or the jack) alternately to the maximum or minimum pressure.

Figure 6-4 Laboratoire Regional Des Ponts et Chaussées - Variable Confining Pressure Apparatus (150 mm x 70 ∅mm)



6.6.3 Variable Confining Pressure Apparatus (320 mm x 160 ∅mm) for Testing Unbound Granular Materials

The two distributors are guided by a current impulse delivered by a timing unit, which provides the time of loading and unloading. The load and cell pressure signals (shown on an oscilloscope) are adjusted to approximate a sinusoidal shape by delivery control valves. In variable confining pressure mode, the frequency of the loading is 0.5 Hz. In constant confining pressure mode, the frequency is 1 Hz.

<u>Specimen Manufacture</u>

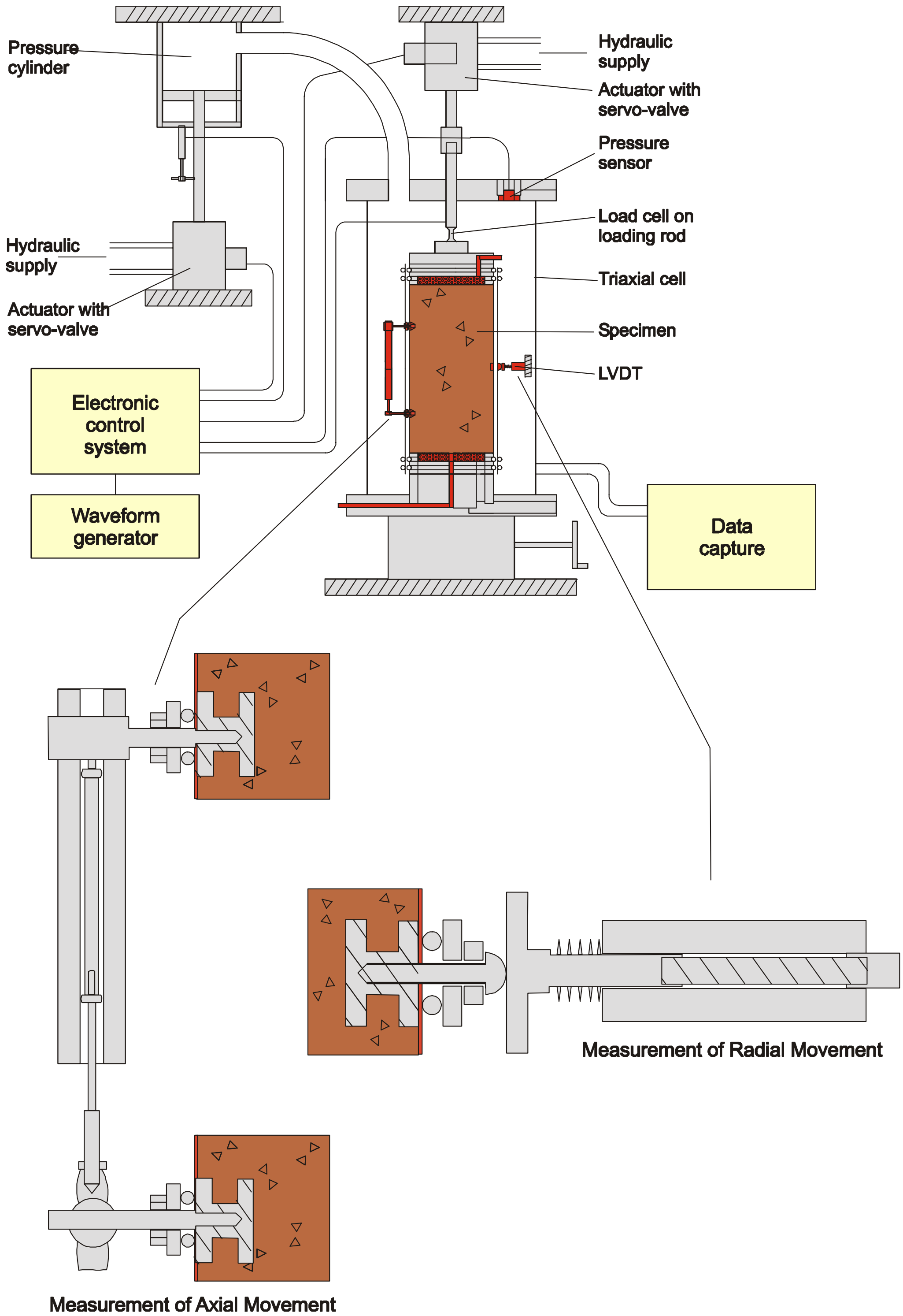
A latex membrane and the attached anchors for the axial and radial measuring devices are placed inside a split mould The specimen is compacted in a single layer, by a full-faced surcharge, while the mould is vibrated, to a specified height and thus dry density. In certain cases the moisture content can be measured to achieve the correct density and subsequently dried. The density of each specimen is checked for uniformity with a radiometric device.

<u>Instrumentation and Data Capture</u>

The axial movement is measured by three LVDTs positioned at 120° to one another, attached between the anchors at $^1/_3$ and $^2/_3$ of the specimen height. These LVDTs are held in place by an aluminium hoop and are supported by the specimen. Three LVDTs, at 120° to one another, measure the radial movement. These are mounted on a Perspex ring at mid-height of the specimen. On the end of the core of the LVDT is a flat disc that is held, by a light spring, to a dome that is attached to the anchor embedded in the specimen. This allows the radial deformation to be measured despite axial displacement of the dome. This is shown graphically in Figure 6-5. All data is collected by means of a computerised data acquisition system.

During tests on unbound granular materials, to maintain a constant air pressure within the specimen, non-woven geotextile discs, treated with a silicone emulsion are interposed between the specimen and the porous stone discs. Permeable to air, these discs allow the interstitial air to be connected with the atmosphere, but allow the pore water suction to be independently controlled by using ceramic discs at the base of the specimen with an appropriate air pressure entry.

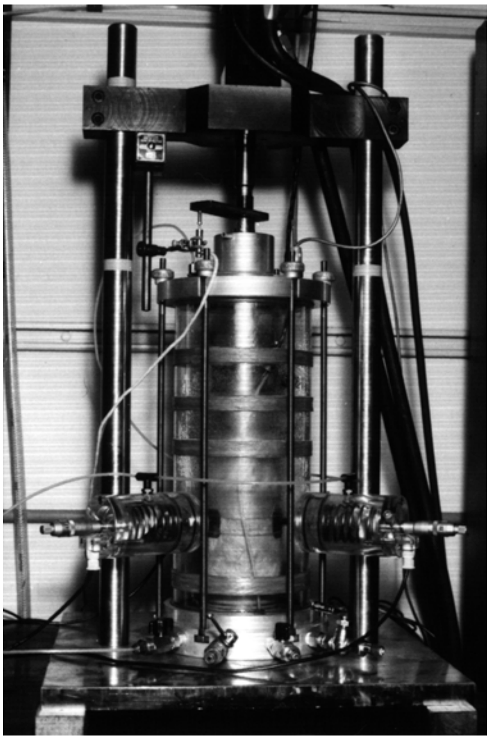
Figure 6-5 Laboratoire Regional Des Ponts et Chaussées -Variable Confining Pressure Apparatus (320 mm x 160 ∅mm)



6.7 DELFT UNIVERSITY OF TECHNOLOGY

Delft University of Technology has two repeated load triaxial apparatus, one for testing subgrade soils and the other for testing unbound granular materials, as shown in Photograph 6-4.

Photograph 6-4 Apparatus at Delft



Constant Confining Pressure Apparatus (200 mm x 100 ∅mm) for Testing of Subgrade Soils

Constant Confining Pressure Apparatus (800 mm x 400 ∅mm) for Testing Unbound Granular Materials

6.7.1 Constant Confining Pressure Apparatus (200 mm x 100 ∅mm) for Testing of Subgrade Soils

<u>The Apparatus</u>

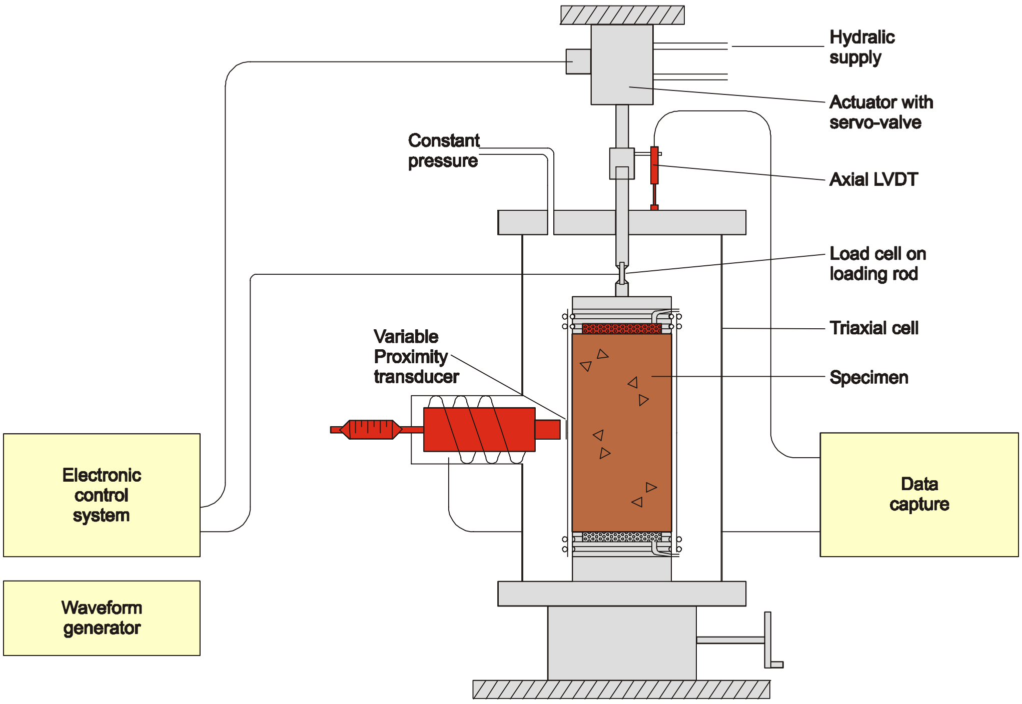
This triaxial test apparatus was developed for investigating the resilient behaviour of finer graded sands and laterites {Sweere (1980)}. The specimen size is 200 mm high and 100 mm diameter. The constant confining stress is applied through air pressure in a Plexiglas cell, whilst the deviator stress is applied by means of a servo hydraulic actuator through a loading piston. This apparatus is shown schematically in Figure 6-6.

A load cell is incorporated inside the triaxial cell, thereby eliminating load measuring errors caused by the friction between the loading piston and the top of the triaxial cell.

<u>Specimen Manufacture</u>
The specimen is compacted in six layers in a split mould with a rubber membrane placed inside it using a tamping compaction device developed at TUDelft.

<u>Instrumentation and Data Capture</u>
Axial deformation is measured by an LVDT connected to the loading piston outside the triaxial cell and thus the end effect of the contact of the specimen and the platens is not eliminated. Radial deformation of the triaxial specimen is measured by three non-contacting PT sensors, which are mounted through the Plexiglas cell on a horizontal plane at mid-height of the specimen.

Figure 6-6 Delft University of Technology - Constant Confining Pressure Apparatus (200 mm x 100 ∅mm)



6.7.2 Constant Confining Pressure Apparatus (800 mm x 400 ∅mm) for Testing Unbound Granular Materials

<u>The Apparatus</u>
This apparatus was developed in the 1980s for testing of unbound granular materials for roads {Sweere (1990)}. Due to the large specimen size of 800 mm in height and

400 mm in diameter it was necessary to try and circumvent the need for a triaxial cell around the specimen. In a similar method to that used by LNEC a sub-atmospheric pressure is applied within the membrane to the triaxial specimen. In this manner a confining stress is simulated.

The specimen is enclosed by a double membrane, which has an airtight connection to the bottom and top platens using 'O-rings' and grease. A constant vacuum is applied to the specimen through a bore in the top platen, while another bore in the top platen is connected to a vacuum reducer providing a constant controlled bleed. The hydraulic system can apply 100 kN (800 kPa) repeatedly, at frequencies of up to 10 Hz. This apparatus is shown in Figure 6-7.

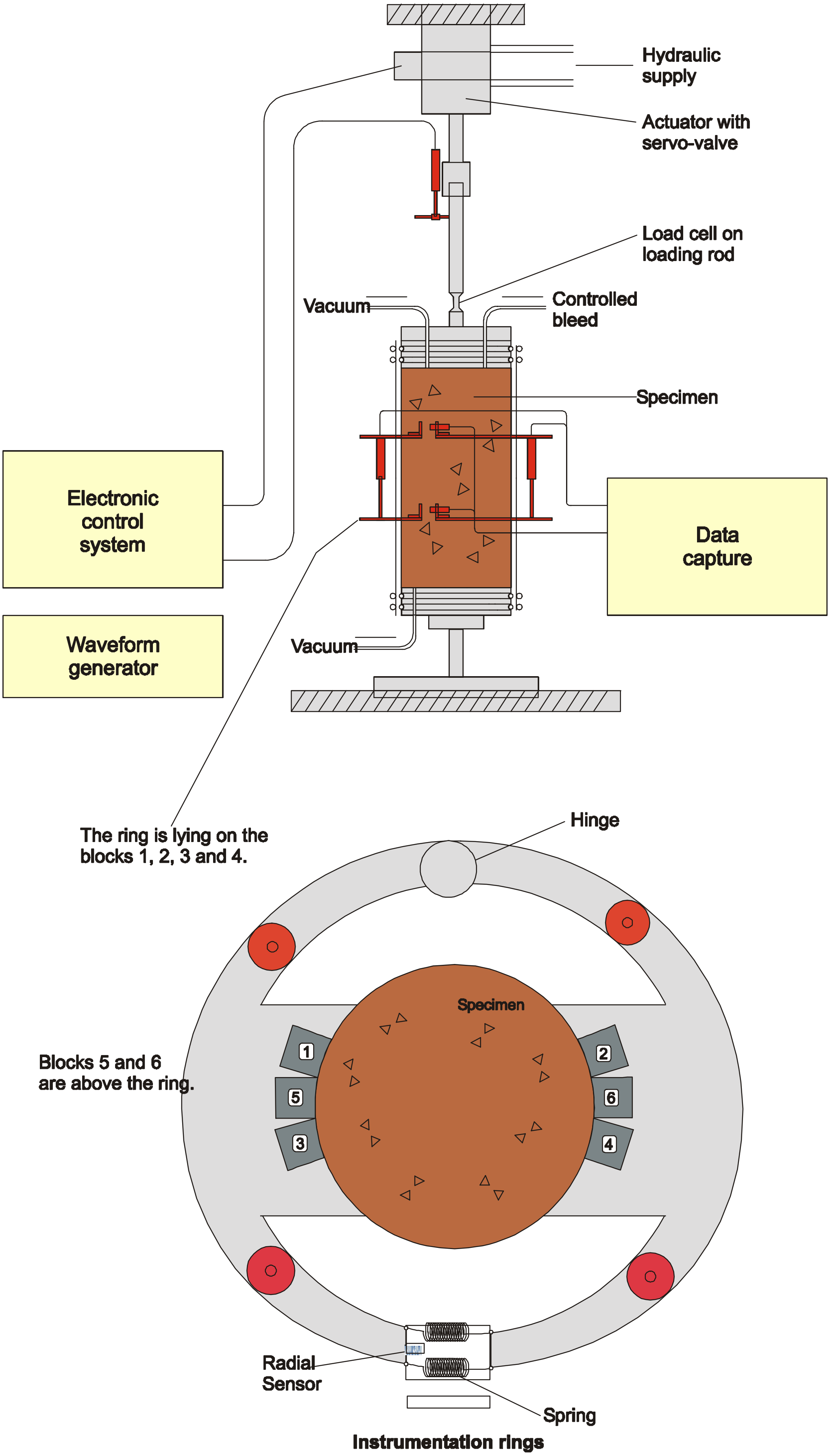
Specimen Manufacture

The material is compacted in a split mould lined with a membrane made from plastic of 0.4 mm thickness, which are welded into on oversize barrel shape to allow the specimen to expand in a radial direction under load. The material is divided into 8 portions compacted into the mould with a heavy tamper. A full face compactor plate is applied to the second, fourth, sixth and eighth layers for 30 minutes each using a haversine load of 7 Hz frequency and 40 kN amplitude superimposed on a 10 kN static load. Due to the large amount of material required to make a specimen it was found not to be possible to control the moisture content accurately.

Instrumentation and Data Capture

LVDTs mounted on two Plexiglas rings surrounding the specimen, at positions of one third and two thirds of the height, and measure the axial and radial deformations of the specimen. Plastic blocks glued to the membrane support these rings, thus the specimen supports the instrumentation. Four vertically mounted LVDTs between the rings measure axial deformations, while two horizontally mounted proximity transducers are used to measure the radial deformations as the rings open and close. This is shown in detail in Figure 6-7.

Figure 6-7 Delft University of Technology - Constant Confining Pressure Apparatus (800 mm x 400 ∅mm)



6.8 COMPARISON OF THE APPARATUS AND INSTRUMENTATION SYSTEMS

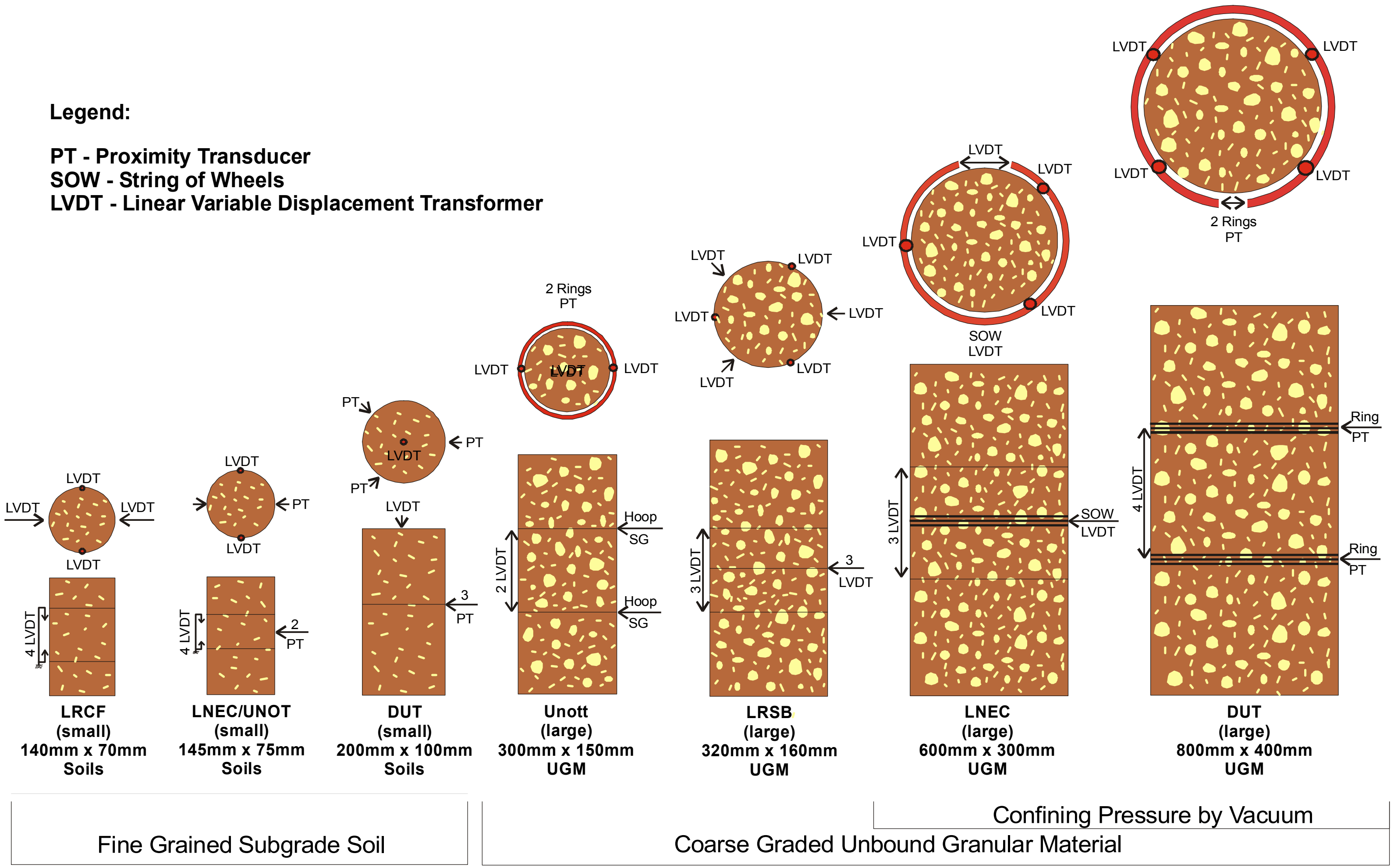
In total eight apparatus were used in the five laboratories and seven of these differ to some degree. The specimen diameter varies from 70 mm to 400 mm between the laboratories as illustrated (diagrammatically) in Figure 6-8.

In all apparatus a load cell attached to the upper loading piston measures the axial load. It is noted that the measurement of the actual loads applied to the specimen should be made inside the cell, where relevant, thus avoiding the friction caused by the plunger and the cell top, which can be quite substantial since a good seal is required due to pressurisation of the cell.

There are two methods of applying radial stress, the first, used on the smaller specimens, is to apply a confining pressure within the cell using either non-conductive silicone oil (or air in the case of LRSB). In general these systems allow the radial stress to be applied cyclically. The second method is to apply a constant vacuum to the specimen, which is enclosed in an airtight membrane attached to the top and bottom platens, using a vacuum pump. The second system does not allow repeated loading of the radial stress.

To obtain representative strain values instrumentation is required which will be affixed to the specimen somewhat remote from the ends and which will measure deformation longitudinally and radially as the specimen is loaded. Figure 6-8 indicates the instrumentation systems used at each laboratory on their repeated load triaxial equipment and these are summarised in Table 6-1. There are obvious differences between these instrumentation systems but primarily size and weight are important. Since the influence of the weight of the instrumentation on the specimen is more critical on the smaller specimens these tend to have a method of supporting the instrumentation remote for the specimen whereas the larger, stronger, specimens are capable of supporting the instrumentation on the specimen.

Figure 6-8 Instrumentation Layout for the Repeated Load Triaxial Apparatus



The three apparatus designed for testing fine-grained subgrade soils (LRCF, UNOT and LNEC) are almost identical; however there is more difference in the remaining four apparatus (which are designed for testing larger unbound granular base material). The largest two specimens (LNEC and DUT) provide a confining pressure by internal vacuum rather than by a pressure chamber and thus the size of their on-sample instrumentation systems are less restricted.

6.8.1 Instrumentation Fixing Methods

Axial Strain Measurement Systems

In all cases, the axial strains were measured using LVDTs as shown in Table 6-1. In the case of the smaller systems (LRCF, LNEC/ UNOT), two pairs of LVDTs bodies are supported on either side of the specimen, on a frame fixed to the base plate of the triaxial cell, and only the armatures are carried by the sample. Small cruciform vanes are inserted into the wall of the triaxial specimen vertically above each other, at the quarter points (LRCF) and the third points (UNOT/ LNEC) of specimen height, and the membrane is placed over the specimen. A pin is fixed to the vane, by piercing the membrane, and also to the armature of the LVDT. A disadvantage of this arrangement is that the axial strain reading comes from the difference between two measures and thus includes a greater error probability than if read as one measurement. However, since the total weight of this system is <5 g, this minimises the stress imposed by the weight of the on-sample instrumentation and thus the likelihood of specimen deformation.

The smaller apparatus at DUT does not use on-sample instrumentation for axial strain measurement but measures the axial deformations from the top platen but this has been shown to give erroneous results due to end effects between the specimen and the platens {Sweere (1990)}.

For three of the four larger unbound granular material systems one LVDT was made to span between fixings at either the third (LRSB, DUT large) or quarter points (UNOT large). This is because the stronger specimens are assumed to be more able to carry the weight of the complete instrument.

Table 6-1 Summary of Triaxial Apparatus of the Participating Laboratories

Laboratory	Specimen Size (mm)		Strain Measurement		Loading System	
	Height	Diameter	Axial	Radial	Deviatoric	Confining
LNEC[1,6]	150	76	2 pairs LVDT on studs[3]	2 proximity transducers	Servo-hydraulic	Servo-hydraulic
LNEC[7]	607	308	2 LVDTs on glued blocks	'String of Wheels'	Servo-hydraulic	Partial vacuum, non-repetitive
UNOT[6]	150	76	2 pairs LVDT on studs[3]	2 proximity transducers	Servo-hydraulic	Servo-hydraulic
UNOT[7]	300	150	2 LVDTs on studs	2 flexible hoops on studs	Servo-hydraulic	Servo-hydraulic
LRCF[2,6]	140	70	2 pairs LVDT on studs[3]	2 pairs LVDT to studs[5]	Servo-hydraulic	Servo-hydraulic
LRSB[7]	320	160	3 LVDTs sprung to studs	3 LVDTs to domed studs[5]	Pneumatic	Pneumatic
DUT[2,6]	200	100	External LVDT	3 proximity transducers[5]	Servo-hydraulic	Hydraulic, non-repetitive
DUT[7]	800	400	4 LVDTs between hoops	2 calliper hoops on glued blocks[4]	Servo-hydraulic	Partial vacuum, non-repetitive

Notes:
1 Can measure and control pore suctions
2 Can measure pore suctions
3 Each LVDT body is mounted on a fixed frame.
Axial strain is computed from the difference between readings of a pair mounted above each other
4 Each calliper touches the specimen at two, opposite, points
5 Adjustable through cell wall during testing
6 Primarily for fine-grained subgrade soils
7 Primarily for base and subbase aggregates

For the larger unbound granular material specimens at UNOT and LRSB studs are placed in the specimen's material prior to compaction. Again, a membrane is placed over the specimen and a pin pierces the membrane, which is then sealed. For the much larger triaxial specimens at LNEC and DUT the fixing is provided by a block, glued to, but not penetrating, the plastic membrane. Glueing allows easy attachment of the fixing after the sample has been compacted and placed in position, ready for testing. Also, there is no possibility of the fixing affecting the local quality of the compaction of the material, which may cause unreliability with strain measurements when embedded fixings are utilised.

On the LRSB apparatus the LVDTs have cones at each end that are spring-loaded into a cup attached to the fixing. This approach enables the sensor to measure the spacing between two points (which are arranged to be in the periphery of the specimen) with minimal influence of the rotation of the fixings that might occur due to the influence of individual aggregate particles, for example.

The larger UNOT apparatus has simple threaded rods to which the LVDTs are attached. It has the advantage of greater simplicity but the influence on axial strain measurement of rotation of the fixings may not be negligible. The large DUT apparatus clamps the LVDT to a Plexiglas ring that is fixed to the glue-on blocks. A rod with two universal joints extends the LVDTs armature, which allows for any lack of coaxiality in the fixing points.

<u>Radial Strain Measurement</u>

Three of the apparatuses use proximity transducers to measure radial strain as shown in Table 6-1, in which a piece of aluminium foil is placed between specimen and membrane as a ‘target’ (LNEC, UNOT and DUT small). For radial strain measurement the smaller apparatus at LRCF and larger apparatus at LRSB use embedded studs similar to those used by LRSB for axial strain measurement fixings into which domed nipples are screwed. Spring-loaded LVDTs set in the cell wall have plate tips that rest on the domes thus measuring the radial movement allowing some longitudinal movement. These are positioned at one third and two thirds of specimen height and at 180° (LRSB) and 120° (LRCF) spacing around the specimens.

On the small triaxial apparatus at UNOT and LNEC these transducers are supported on the same frame that carries the bodies of the axial LVDTs. Strains are thus measured only at mid-height of the specimen with two opposing sensors. At DUT they are held in the cell wall of the smaller apparatus at mid-height of the specimen (with 3 sensors at 120° pitch around the specimen) but, in the larger DUT apparatus, are affixed to a calliper Plexiglas ring that rests on some of the glue-on blocks. A proximity transducer acts across the opening jaws of the hinged callipers. In that apparatus there are two such rings enabling the radial strain to be assessed at both one third and two thirds of the specimen height.

In the large LNEC apparatus a necklace-style ‘string-of-wheels’ is used at mid height with a LVDT as the active part across an opening. This ‘necklace’ comprises a steel cable, drawn tight around the specimen by a sprung link across the opening in the ‘necklace’, on which are arranged 12 ‘single-axle bogies’. The wheels thus keep the cable a constant distance from the specimen or, strictly speaking, the membrane necessitating an increased opening in the ‘necklace’ as the specimen expands. An LVDT is placed across this opening to monitor strain.

The large UNOT apparatus uses epoxy resin hoops fitted to the same fixings that hold the axial LVDTs, thus providing strain measurements at one quarter and three quarters of specimen height. The hoops are strain-gauged using resistance wire. As the sample expands the curvature of the hoop changes and this is sensed by a change in the resistance of the gauges.

Because the specimen did not require a membrane the effect of a membrane and the glue-on fixing methods used at LNEC and DUT on their large vacuum confining pressure apparatus was not tested. In a separate study, Karaşahin (1993) compared the performance of the UNOT LVDT and epoxy hoop system, which is supported on inserts or studs, with the same instruments supported on glue-on fixings. He observed that, while the confining stress remained constant, the two instruments gave comparable readings. However, if the confining stress changed the two systems gave very different results with much higher radial strains and lower axial strains with increasing confining stress when glue-on fixings were used.

Cheung (1994) tested a smooth-sided rubber specimen to check whether glue-on instruments might misread due to slipping of the membrane. Even when applying only a partial vacuum confining pressure of 10 kPa, he detected no significant effect on resilient axial or radial strain measurements.

6.9 Phase 4 - Instrumentation Comparison on the Artificial Specimen

As described in an earlier chapter there are five test phases conducted during this work. An experiment to compare six sets of different displacement measuring instruments attached to the same artificial specimen was called Phase 4, the other four phases are described in the next chapter. This experiment was undertaken using the LRSB variable confining pressure apparatus to apply the loading. The specimen comprised Polytetrafluoroethylene (PTFE), which is a visco-elastic material but has a fairly linear stiffness with stress, and was 160 mm diameter and 320 mm tall. This implies that the specimen response under loading depends on the loading time and possibly the waveform of the generated loading signal. In order to eliminate this influence a static load regime was prescribed in the form of a square-wave loading. Also, a conventional repeated loading regime was applied at a range of stresses. The stress regimes are shown in Table 6-2 and Table 6-3 and the instruments considered for this experiment are listed in Table 6-4.

Table 6-2 Static Stress Regime applied during Instrumentation Comparison

Stress Regime Name	Time (minutes)	Confining Pressure [σ_3] (kPa)	Deviator Stress [q] (kPa)	Stress Ratio [q/p]
Static Radial Test 1 (SR1)	60	250	300	0.9
	60	0	0	
Static Radial Test 2 (SR2)	60	250	375	1.0
	60	0	0	
Static Radial Test 3 (SR3)	60	100	200	1.2
	60	0	0	
Static Axial Test 1 (SA1)	60	100	150	1.0
	60	0	0	

Table 6-3 Dynamic Stress Regime applied during Instrumentation Comparison

Stress Regime Name	No.of Cycles	Confining Pressure [σ_3] (kPa)		Deviator Stress [q] (kPa)		Stress Ratio [q/p]
Dynamic Radial Test 1 (DR1) and Dynamic Radial Test 2 (DR2)	100	Min	0	Min	0	1.36
		Max	100	Max	250	
	100	Min	0	Min	0	1.20
		Max	100	Max	200	
	100	Min	0	Min	0	1.00
		Max	100	Max	150	
	100	Min	0	Min	0	0.75
		Max	100	Max	100	
	100	Min	0	Min	0	0.43
		Max	100	Max	50	
	100	Min	0	Min	0	0.23
		Max	100	Max	25	
	100	Min	0	Min	0	0.00
		Max	100	Max	0	

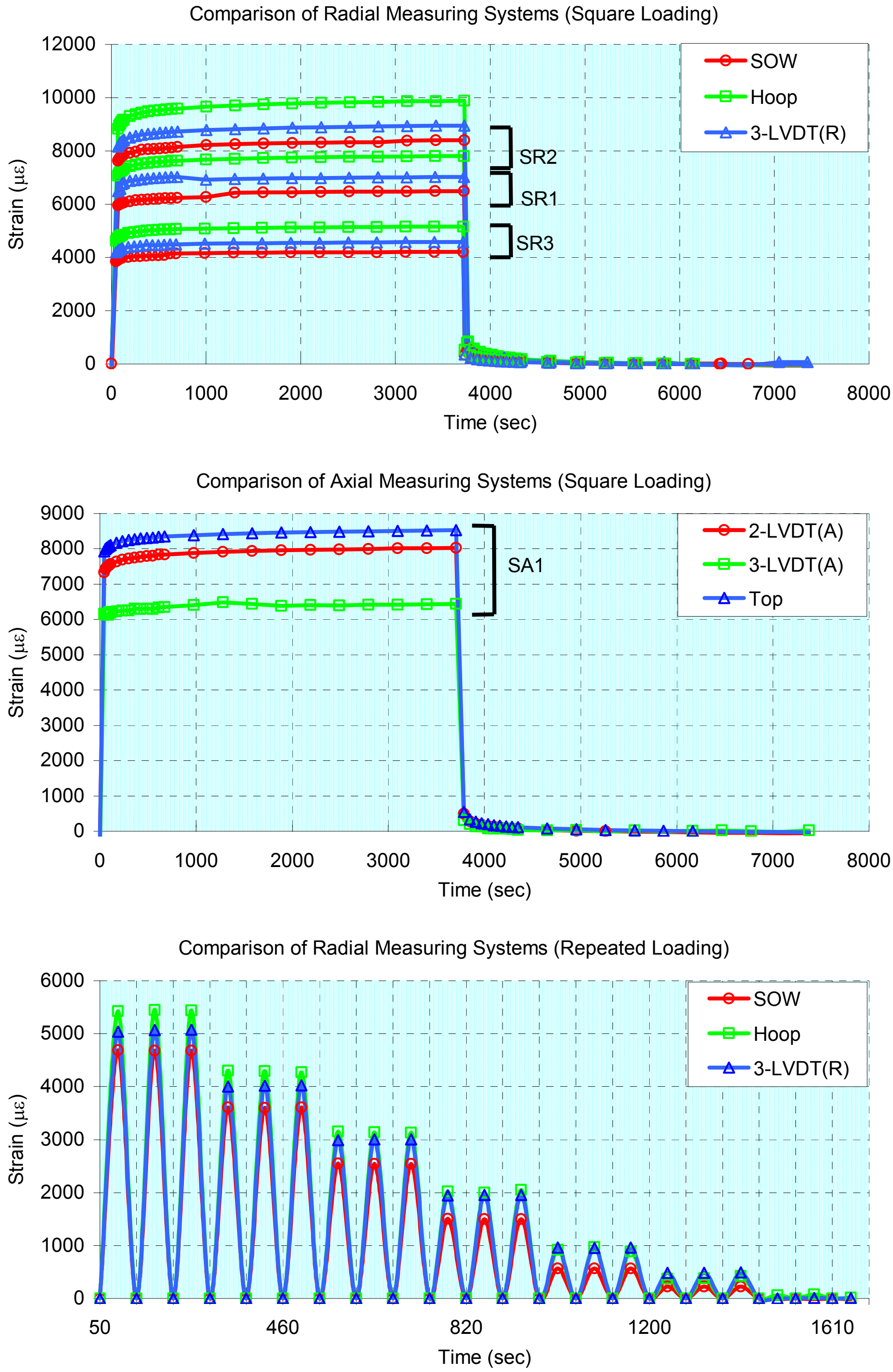
Table 6-4 Instrumentation Tested during the Single-Specimen Comparison

Apparatus	Height and Diameter (mm)	Instrumentation	Abbreviation
LNEC	600 x 300	String-of-wheels for radial strain[1]	SOW
UNOT	320 x 160	LVDTs on stud and rod system for axial strain	2-LVDT (A)
UNOT	320 x 160	Strain-gauged epoxy hoops on common stud and rod system	Hoop
LRSB	320 x 160	3 LVDTs spring loaded into cone and cup fittings for axial strain	3-LVDT (A)
LRSB	320 x 160	3 LVDTs acting radially, mounted in cell wall	3-LVDT (R)
DUT	210 x 102	1 LVDT to end platen for axial strain	Top

Note:
1 – Scaled down model manufactured.
(A) – Axial; (R) - Radial

A graphic representation of the results is shown in Figure 6-9. Complete results from each stress path applied during the experiment are contained in Appendix C.

Figure 6-9 Instrumentation Comparison showing differing Strain and Stress Conditions



Note: These results are for static tests

Radial Systems

For the three radial strain systems the LRSB system of 3 radial LVDTs is closest to the mean with the hoop consistently recording greater strains and the SOW recording small strains. This scatter is approximately ±10% about the mean value as shown in Table 6-5.

The dynamic radial test results clearly show (Table 6-5) that the accuracy is dependent on the magnitude of the movement measured. These instruments can be highly inaccurate for small strain measurements, however with the development of better electronic instruments it is likely that these problems will be overcome in the future.

Table 6-5 Instrumentation Comparative Results on Artificial Specimen

Static Radial Test (Square-Wave Loading)

Test	Confining Pressure (kPa)	Deviator Stress (kPa)	Stress Ratio q/p	Strain (με)				Deviation (%)		
				SOW ε3	Hoop ε3	3-LVDT (R) ε3	Mean	SOW ε3	Hoop ε3	3-LVDT (R) ε3
SR1	249	299	0.9	6496	7813	7020	7110	-9%	10%	1%
SR2	249	374	1.0	8372	9870	8941	9061	-8%	9%	1%
SR3	100	200	1.2	4207	5161	4579	4649	-10%	**11%**	2%

Static Axial Test (Square-Wave Loading)

Test	Confining Pressure (kPa)	Deviator Stress (kPa)	Stress Ratio q/p	Strain (με)				Deviation (%)		
				2-LVDT (A) ε1	3-LVDT (A) ε1	Top ε1	Mean[2]	2-LVDT (A) ε1	3-LVDT (A) ε1	Top ε1
SA1	100	150	1.0	8047	6410	8519	7228	**11%**	**-11%**	**18%**

Dynamic Radial Test (Repeated Loading)

Test	Confining Pressure (kPa)	Deviator Stress (kPa)	Stress Ratio q/p	Strain (με)				Deviation (%)		
				SOW ε3	Hoop ε3	3-LVDT (R) ε3	Mean	SOW ε3	Hoop ε3	3-LVDT (R) ε3
DR1	99	249	1.4	4684	5440	5060	5062	-7%	7%	0%
DR2	99	198	1.2	3605	4288	4014	3969	-9%	8%	1%
DR3	100	149	1.0	2545	3140	3001	2895	**-12%**	8%	4%
DR4	100	98	0.7	1501	2022	1950	1824	**-18%**	**11%**	7%
DR5	100	48	0.4	566	925	958	817	**-31%**	**13%**	**17%**
DR6	100	23	0.2	225	391	486	367	**-39%**	6%	**32%**
DR7	100	4	0.0	-13	51	5	15	**-186%**	**253%**	**67%**

Note: 1. Red bold text denotes deviation values that exceed 10% from the mean.
2. The mean values exclude the DUT Top measurement.

Axial Systems

For the three axial strain systems used the mean did not include the external top platen LVDT used because this is known to give erroneous readings due to the end effects of the specimen. Differences of about ±10% about the mean strain were observed for the other two systems.

6.10 INSTRUMENTATION LIMITATIONS

Repeated load triaxial testing requires instruments that remain at a near constant sensitivity over a large range because testing often includes the measurement of permanent as well as resilient deformations. Therefore the same instrument collects small resilient strains even after some (relatively large) permanent deformation has taken place. LVDTs, PTs and strain-gage transducers all give a continuous signal over their range and thus, in principle, are infinitely discriminatable. In practice, current digital data acquisition systems limit this to the strain required to generate ±1 bit {Dawson and Gillett (1998)}. For example if a maximum permanent deformation of 6% is expected, a 16 bit system gives a theoretical discrimination of $\approx \pm 1\mu\varepsilon$ ($= 0.06/2^{16}$) whereas an older 12 bit system would only yield a discrimination of $\approx \pm 15\mu\varepsilon$. Digital noise generally means that figures twice as large have more realistic discrimination capabilities. A further loss of discrimination by a factor of 3 would be needed for a very soft soil for which a 20% strain failure test was to be monitored by the same equipment. Thus between $\approx \pm 6\mu\varepsilon$ and $\approx \pm 90\mu\varepsilon$ discriminations would apply for the instruments used in this work.

The proximity transducers operate over a limited range of deformation, requiring a small gap relative to their size. Those with a large range are also themselves large and thus give rise to problems fitting them into a triaxial cell. Some non-linearity may also be introduced since the curvature of the specimen wall (which carries the target) will be more significant for a large sensor than for a small one. For this reason small proximity transducers may be used but removed after initial strain is complete as the specimen swells and threatens to touch them {Dawson and Gillett (1998)}; or mounted through the cell wall so that an external coarse adjustment may be made as the test proceeds (the small DUT device solution).

LVDTs find the requirement of range and sensitivity no problem, and if they are deformed beyond the expected range the armature can (normally) slide far beyond its operational limit and no damage results.

Resistance wire strain gauges occupy a middle ground as they sustain damage if grossly over-deformed. In the context of likely strains in repeated load triaxial tests excessive deformation is not usually a problem. However they do present an environmental problem, unless very carefully shielded (a difficult task on very small lightweight instrumentation) they will often pick up extraneous noise, thus limiting the possible discrimination.

Most of these instruments are not waterproof and thus must be used in non-aqueous conditions such as air for constant confining pressure or a non-conducting fluid such as the silicone oil for the application of cyclic cell pressures. Adjusting instruments contained in the cells, particularly those using fluids, is messy and requires pressures to be removed. Since the confining stress for the largest specimens (DUT and LNEC) is a vacuum, instrumentation does not need to be fully waterproof, and the absence of a confining cell allows instrumentation to be adjusted mid-test.

6.11 ASSESSING INACCURACIES IN LABORATORY TESTING OF MATERIALS

All measurements in physics and science are generally inaccurate to some degree. There exists, however, an accurate result whereby the deviation from the actual value is considered insignificant, for the purposes required and this is thus acceptable.

During testing, subsequent reporting of the results and the use of the results in pavement design, it is imperative that the designer has confidence in the laboratory results. The designer should have a good understanding of the magnitude of the errors involved, in the determination of the results since any errors will be carried forward to the design. It is good practice that an error analysis be conducted in order that a sound economic pavement design will result.

6.11.1 Identifying Errors

The difference between the observed value (that is recorded) and the genuine value (some real value that remains unknown) is called the error of observation. Obviously when testing a road construction material an aim would be to minimise the errors in the results as much as possible. However, this minimisation of errors need only be enough so that the errors in the measurement are insignificant enough so as not to affect the conclusion inferred from the results. It is thus possible that a crude test may yield results, which will serve the purposes well enough.

Errors may be grouped into two categories namely accidental or systematic. It is often difficult to distinguish between these two types of errors, particularly since many inaccuracies occur due to a combination of the two categories:

- Accidental Errors — These errors are frequently due to the limitations in control of the equipment and accuracy of the instrumentation. They also may be due to different operators, apparatus, machine induced variations in material properties, specimen preparation, specimen instrumentation and variations in test procedures.

  These errors may be identified when two tests are conducted using a single specimen, instrumentation and apparatus i.e. repeated observations. They are disordered in their incidence and variable in magnitude while occurring in no ascertainable sequence.

- Systematic Errors — Systematic errors may arise from the operator or the equipment and repeated observations do not necessarily reveal these errors. These errors are repeated over and over again for different tests and even, when these errors are identified, they are sometimes difficult to eliminate. A systematic error, for example, may result from testing at room temperature that is different from that in the field.

6.11.2 Errors Occurring During the Manufacture of the Specimen

Once a material sample is divided into the correct fractions, and the specified moisture content attained, the operator compacts the specimen in a mould to a specified density. Methods of compaction vary greatly between laboratories. The compaction method depends on the specimen size, number of layers, moisture content and sophistication of the compaction equipment.

The uniformity of material specimens can be identified and corrected fairly easily by checking the density of the specimen and the moisture content. Of course the uniformity throughout a specimen is just as important as the uniformity between different specimens. It is possible to check specimen uniformity along the specimen length using nuclear density test apparatus {Gomes Correia (1985)}.

Assuming the specimens' fall within the tolerances specified for density and moisture content, the specimens may now be mounted in the triaxial apparatus and the strain measurement instrumentation attached. The test apparatus and instrumentation vary greatly between laboratories and consequently it is often difficult to follow set procedures exactly, due to differing apparatus and operators.

6.11.3 Errors Occurring During the Repeated Load Triaxial Testing

In order that repeated load triaxial testing produces the required results or material parameters, it is necessary first to prepare a specimen, or specimens, to a specified standard (for example moisture content and density). The specimen is then mounted in a test apparatus that must be capable of applying stresses to a defined specification that is within a particular accuracy. Instrumentation that measures the movement of a specimen, under the specified load regime, must be able to record the deformation experienced by the specimen to a specified accuracy. Within these operations there are bound to be errors. Such errors follow no simple law and arise from many causes.

Different laboratory operators conducting a test to a strict procedure will certainly conduct the tests slightly differently. This will result is some difference in results. In a single laboratory there may exist more than one apparatus, each producing slightly different results. Similarly there will be differences produced by the recording instrumentation used.

6.11.4 Errors Occurring During the Analysis of the Results

Having obtained the stresses and strains from the laboratory testing, it is necessary to conduct some analysis in order that the material properties (characteristics) required for the pavement design are attained. It is at this point that any differences between specimens etc. will become evident. It is not always obvious, however, which are 'bad' results and which are 'good'. A possible solution is that a range of values are produced which encompass any probable errors so that the final pavement design can be conducted with a high degree of confidence. This would be the worst case design and would be part of a sensitivity analysis. The best outcome of this procedure would be where the design using the worst values was no more expensive than that using the best values. This would indicate that there were no significant errors.

6.12 Basic Statistics

During this work, in identifying errors, it has been necessary to include some statistical analysis on the results obtained from the testing. Every attempt has been made to keep this as simple as possible as described in the following section.

The degree to which numerical data tends to spread about an average value is called *dispersion*, or *variation*, of the data. Taking a set of numbers, there exist a *range* within these number, which is defined as the difference between the largest and the smallest numbers in the set. There is also a *mean*, or *average*, of the numbers in the set. The *deviation from the mean*, or *deviation* of any single value is the difference between the absolute value (always positive) of the number and the mean.

The *standard deviation* in the set of numbers is an indication of the variation of the all of the numbers, within the set, from the mean.

The *coefficient of variation* of a set of numbers is defined as the standard deviation divided by the mean and is expressed as a percentage herein.

Subgrade Soils

For each model the variables as described above are calculated for the test data. A small report generated by NLREG, the software computer programme that is used to analyse the subgrade soil results {Sherrod (1998)}, that lists each variable, the minimum and maximum value and the mean and standard deviation of the variable data.

Calculated Parameter Values

For each parameter of each model the initial parameter estimate (which is generally taken as 1) is shown in the computer report. Also, the final parameter estimate, the standard error of the estimated parameter value as well as the 't' and the Prob(t) values. The significance of these statistical values is discussed in turn:

The Final Parameter Estimate

NLREG uses a model/ trust-region technique along with an adaptive choice of the model Hessian. The algorithm is essentially a combination of Gauss-Newton and Levenberg-Marquardt methods, however, it is claimed by Sherrod (1998) that the adaptive algorithm often works better than both of these methods.

The basis for the minimisation technique used by NLREG is to compute the sum of the squared residuals for one set of parameter values. Each parameter value is then slightly altered and the sum of squared residuals recomputed to see how the parameter value change affects the sum of the squared residuals. By dividing the difference between the original and new sum of squared residual values by the amount the parameter was altered, NLREG is able to determine the approximate partial derivative with respect to the parameter. This partial derivative is used by NLREG to decide how to alter the value of the parameter for the next iteration.

Sherrod (1998) stated that when the modelled function is 'well behaved', and the starting value for the parameter is not too far from the optimum value, the procedure will eventually converge to the best estimate for the parameter. This procedure in NLREG is carried out simultaneously for all parameters and is, in fact, a minimisation problem in n-dimensional space, where 'n' is the number of parameters.

The Standard Error of the Estimated Parameter

The standard error values that are associated with computed parameters give an indication of how exact the estimated value is: the smaller the standard error, the more confident one can be that the actual value of the parameter's value matches its estimated value. It is somewhat similar to taking a sample from a large set of observations and computing the mean. In that case the standard error, or standard deviation, of the mean indicates how likely the sample mean matches the mean of the entire set that is being sampled. In the case of a function with multiple parameters there is a separate standard error value for each parameter because the confidence and accuracy of each estimated value may be different.

The 't' Statistic

The 't' statistic is computed by dividing the estimated value of the parameter by its standard error. This statistic is a measure of the likelihood that the actual value of the parameter is not zero. The larger the absolute value of t, the less likely that the actual value of the parameter could be zero

The Prob(t) Value

The Prob(t) is defined as the probability of obtaining the estimated value of the parameter if the actual parameter value is zero. The smaller the value of Prob(t), the more significant the parameter and the less likely that the actual parameter value is zero. For example, assume the estimated value of a parameter is 1.0 and its standard error is 0.7. Then the t value would be 1.43 (1.0/0.7). If the computed Prob(t) value was 0.05 then this indicates that there is only a 0.05 (5%) chance that the actual value of the parameter could be zero. If Prob(t) was 0.001 this indicates there is only 1 chance in 1000 that the parameter could be zero. If Prob(t) was 0.92 this indicates that there is a 92% probability that the actual value of the parameter could be zero; this implies that the term of the regression equation containing the parameter can be eliminated without significantly affecting the accuracy of the regression. One thing that can cause Prob(t) to be near 1.00 is having redundant parameters.

The quality of the fit of one material constant relative to another can be quantified by dividing the standard error by the mean.

Unbound Granular Materials

The model coefficients are calculated for each relationship using a spreadsheet-based method. The correlation coefficient for the particular set of data is calculated based on the experimental data and that predicted by the particular model. It is possible to determine in a qualitative manner how well a model describes the relationship between variables or experimental data. There is a ratio of the explained variance to the total variation that is called the coefficient of correlation (or correlation coefficient). Since this ratio is always positive it is denoted by r^2 and varies between 0 (very poor correlation) and 1 (very good correlation).

6.13 SUMMARY

Of the eight apparatus contained in five laboratories seven vary to some degree. There is a high variability between the instrumentation, which measures the stresses and strains, of the apparatus.

There is no system that clearly stands out above other systems; most systems have been developed because of some needs or preference within the particular laboratory. They all use some form of electronic transducer or strain gauge to measure the movements and stresses and capture the data using an electronic device.

Further, there are many views about the actual fixing of the instrumentation to the specimens; again these vary between laboratories according to preference. It is however, very important that some understanding of the possible errors and inaccuracies of the particular system is undertaken by monitoring and calibration. An example of this is shown by the fact that digital noise was found to account for strain measurements of up to 90$\mu\varepsilon$ during this study.

Sample instrumentation is fixed to the specimens by a number of different methods. Placing measurement studs or pins into the specimen provides a positive method of measurement of axial specimen deflection that eliminates the possibility of slip, which could conceivably occur between a spring-loaded clamp and the membrane. The major drawback associated with using studs or pins in a granular material is that specimen preparation is greatly complicated because of the presence of a stud (or pin), which protrudes both into and out of the specimen. To prepare a granular specimen studs must be affixed to the mould, which can cause problems with the material density around the studs.

The advantages and disadvantages of the various apparatus and instrumentation is summarised in Table 6-6 and Table 6-7 respectively.

Table 6-6 Summary of the Advantages and Disadvantages of Various Instrumentation Methods

Instrumentation	Advantage	Disadvantage
1. Cruciform vanes pressed into soil specimens	On specimen measurement Located at 1/3 or 1/4 height thus no end effects No slippage	Some specimen disturbance Possible rotation of pins if barrelling occurs. However, if cones/domes are used then this is alleviated but a radial stress is applied Studs cause significant specimen disturbance during compaction
2. Studs embedded into UGM specimens during compaction		
3. Instrumentation supported by a frame	Little weight applied to the specimen, particularly important for small soft soil specimens	Doubles the number of transducers required and therefore more than doubles the potential error
4. Axial measurement on the top platen	Easy adjustment to instrumentation	Errors in measurement due to end effects
5. Radial hoops and callipers	No direct attachment to material, therefore no disturbance	Slippage can occur Some radial restraining pressure is applied to the specimen Membrane compression due to cyclic cell pressure causes misreading of radial strain
6. String of Wheels	No direct attachment to material, therefore no disturbance Easily positioning of the instrumentation	Slippage can occur Some radial restraining pressure is applied to the specimen Membrane compression due to cyclic cell pressure causes misreading of radial strain
7. Glued studs or blocks onto the membrane	No direct attachment to material, therefore no disturbance Easily and accurate positioning of the instrumentation	Membrane compression due to cyclic cell pressure causes misreading of radial strain
8. Proximity Transducers	No specimen contact except very lightweight target which is fixed directly to specimen	Small measurement range therefore only useful for radial strain measurement

Table 6-7 Summary of the Advantages and Disadvantages of Various Apparatus Methods

Apparatus	Advantage	Disadvantage
1. Pneumatic cell pressure (Vacuum)	Clean Cheap Instrumentation is assessable for adjustment during a test	Some time lag in loading due to compressible nature of air Rapid variation of the confining pressure is not possible i.e. repeated loading
2. Pneumatic cell pressure within a confining cell	Clean	Some time lag in rapid loading due to compressible nature of air therefore wave shape not very controllable Potentially dangerous
3. Hydraulic cell pressure	Safe Immediate pressure variation, therefore variable confining pressure load applications are possible to specific forms	Expensive Messy
4. Small specimens	Little material requirements Satisfactory for fine grained soils Apparatus less costly	Not satisfactory for materials with large grain size such as unbound granular materials
5. Large specimens	Satisfactory for coarse grained materials such as unbound granular materials with grain size of up to 40 mm	Vast material requirements If confining cells were to be use the apparatus would be very expensive and bulky If vacuum confinement is used, problems of point 1, above, introduced.

There are some guidelines that have been established with respect repeated load triaxial testing:

- The axial load cell should be placed on the loading rod inside the cell in order to avoid the effects of friction between the rod and the cell.
- Variable confining pressure apparatus are desirable but require a cell surrounding the specimen, this makes accessing the instruments during a test difficult.
- Constant confining pressure apparatus are generally used for large specimens (for testing large particle material) and instead of a surrounding cell use an

internal vacuum. Unfortunately the confining pressure cannot be varied but the instrumentation can be easily accessed.

- Axial deformations measured from the top platen give erroneous results due to end effects between the specimen and the platens. Measurement should be taken some distance from the end platens. Commonly this is conducted between one third and two thirds of specimen height or between quarter points. The greater gauge length obtained from quarter points does result in larger deformations, which will result in a more reliably measured strain reading. Three measuring points should be positioned at 120° to one another around the specimen, in order that any discrepancies such as tilting can be detected. However, the laboratories accessed in this work have not always been found to be practical due to extra instrumentation requirements and space within the cell.
- Some axial measuring LVDTs are glued onto the membrane, which surrounds the specimen, while others are attached to studs embedded into the specimen, penetrating the membrane. These two methods of instrumentation attachment did not show great discrepancies with each other.
- The methods of radial measurement are more varied than axial measurement between laboratories. Again, measurement should be taken within the third or quarter height of the specimen, to eliminate the end effects. As with radial deformation measurement, instruments should be positioned at 120° to one another but this has not always been found to be practical.
- Care should be taken if radial measuring apparatuses are glued onto the membrane, which surrounds the specimen, because the changing of pressure, in the cell or within the specimen may cause membrane compression and consequent misreading of radial strain.

The accuracy of the instrumentation used for measuring specimen deformations has a critical role in these tests. As the smallest measured resilient strains are of the order of 100$\mu\varepsilon$ the resolution of the measuring systems should be about 10$\mu\varepsilon$. Systems capable of this will become more common and affordable with time, however it must be noted that instrumentation systems must be checked for faults and calibrated frequently. It may be possible to periodically check the entire test apparatus and instruments using an artificial specimen of known mechanical properties as a reference.

Experiments with an artificial specimen which was tested in the apparatus of at different laboratories, and at one laboratory with multiple instrumentation, have given some confidence that different instrumentation systems can give similar (although not identical) results. Measurements with instrument influenced variability in the ranges of ±5 to ±10% of the mean value should be expected. These artificial specimen tests did not use embedded fixings and these are thought to be a further contributor to differences between instrument outputs, although this could not be assessed completely independently of other variables. For many purposes embedded fixings are preferred as they avoid membrane interaction problems. Some recommendations for selection have been made on the basis of the data gathered, on an assessment of the inherent limitations of the different instrumentation systems and from experience of their use. These differ depending on the type of specimen and triaxial arrangements. Despite the advice offered here it is clear that the 'best' performance will still contain many uncertainties and inexactitudes that are due to a whole range of factors. The value of inter-laboratory comparisons of the type recorded here is high. Systematic errors will be highlighted, procedures crosschecked and quality generally improved.

7 THE TRIAXIAL TEST PROCEDURES AND RESULTS

7.1 INTRODUCTION

Some of the test phases (introduced earlier) were less successful than others. It is recognised that the failure to specify a workable test procedure for the characterisation of road construction materials, for the four participating laboratories, led to a better understanding of the problems in specifying such a procedure. Chapter 6 described the various apparatus and instrumentation used during the Science project and discussed some of their advantages and shortcomings.

As stated earlier a number of typical European road construction materials (unbound granular materials and subgrade soils) were selected, to cover a range of differences in mechanical behaviour under loading and each of the four participating laboratories were to test the material using their test methods and apparatus but following a common test procedure as closely as possible. It was important that the material to be tested at each laboratory was as uniform as possible. In the case of unbound granular materials sieved fractions were combined in a laboratory and dispatched to the other laboratories in order to achieve consistent quality. The reconstituted subgrade soils varied from sand to clay. These materials were conditioned to the required state, sealed and dispatched to the various laboratories, the team members formulated a number of test procedures, and each one was based on the results of the previous test programme.

7.2 OTHER TEST PROCEDURES FOR THE CHARACTERISATION

Briefly, some discussion is made here about test procedures for testing unbound granular materials and soil subgrades {AASHTO (1994), CEN (2000), Australia Standards (1995)} that exist or are currently being developed.

7.2.1 Test Procedures for Granular Materials

<u>Specimen Preparation</u>

All methods recommend that test specimens be between 100 and 150 mm diameter and from 200 to 300 mm high. Based on the rule that maximum particle size diameter ratio must be less than eight, this means that the maximum particles size must be between 12.5 mm and 19 mm. The CEN method states that the specimen diameter should be at least five times the maximum particle size and that the height of the specimen should be twice the diameter, thus 30 mm maximum particles are permitted for a specimen with a diameter of 150 mm, this is nearer real specifications for granular base material as used for road construction worldwide for example CSRA (1985).

Specimen Density and Moisture Content

Methods of compaction vary in that either static (tamping) or dynamic (vibrating) specimen preparation techniques are specified. The CEN specification requires that a specific density, at a particular moisture content, is attained by compacting the material in a series of six to seven layers using a vibrating process, once formed, the specimen is to be given time (3-7 days) to allow the moisture to reach equilibrium within the specimen. It is recommended that the ends of specimens be made smooth by application of fine material to fill surface voids.

Barksdale et al (1990) recommend the following:

Sample Type 1: Crushed rock with maximum particle size 38 mm with 4% fines, well graded compacted to 100% AASHTO T180 density (Modified Proctor)

Sample Type 2: Crushed rock with maximum particle size 32 mm with 10% fines, well graded compacted to 100% AASHTO T180 density (Modified Proctor)

Sample Type 3: Soil aggregate blend with maximum particle size 32 mm with 20% friable soil, well graded compacted to 95% AASHTO T180 density (Modified Proctor)

Sample Type 4: Natural gravel with maximum particle size 20 mm, well graded, plasticity index < 5, compacted to 95% AASHTO T180 density (Modified Proctor)

All specimens are to be manufactured at Optimum Moisture Content (OMC) and then water is introduced to the specimens until saturation conditions are reached.

The Australian method recommends that for granular materials moisture contents of between 60% and 80% of OMC are appropriate. This test procedure states that, for specimens drier than approximately 70% OMC, the drainage is not critical. It is, however, recommended that a moisture sensitivity analysis, with moisture contents up to full saturation, be conducted. This method states that for diagnostic pavement analysis, the *in-situ* moisture conditions or design moisture condition should be applied and the specimen density should be compatible with the compaction curve defined by the specification or *in-situ* condition.

The CEN method recommends that the following moisture contents be attained for the specified number of specimens:

Water Content (%)			**Dry Density AASHTO T180 density (Modified Proctor)**
$W_{OMC-4\%}$	$W_{OMC-2\%}$	$W_{OMC-1\%}$	
No Specimen	1 Specimen	No Specimen	100%
1 Specimen	2 Specimens	1 Specimen	97%
No Specimen	1 Specimen	No Specimen	95%

Where the optimum moisture content is calculated at maximum dry density defined by the modified proctor

Strain Measurements

The Australian method states that for routine practice, off-specimen axial measurement is satisfactory and that the measurement of radial strain is not essential for routine testing, since pavement design models are relatively insensitive to Poisson's ratio.

The CEN method recommends that both axial and radial strain measurements be made on the specimen, in the most accurate way, thus at $^1/_3$ or ¼ of the specimen height.

Applied Specimen Load

Conditioning of the specimen is required, by the CEN method, of 20,000 load applications at defined axial and radial stresses. The Australian method states that preconditioning cycles should be applied; at every stress stage level selected using the stress combination at that stage. The Australian method further states that completion of preconditioning is to be identified when the ninety fifth percentile of permanent strain is unchanged for ten consecutive cycles.

The AASHTO, Australian and CEN methods require that the axial loads should be applied to the specimen for a period of between 0.1 and 3.0 seconds. The waveform can be sine, haversine or rectangular in shape. In general it is recommended that the loads are applied as quickly as the apparatus will allow.

The ARRB method again states that, for routine application, a constant lateral stress is preferred. However, the CEN method allows for both repeated and static cell pressure test methods depending on the sophistication of the available apparatus.

Determination of Resilient Behaviour

All methods recommend that the determination of the resilient modulus must be made via a number of different loading stresses following preconditioning. AASHTO and ARRB state that these should proceed in a descending order of stress ratio whereas the CEN methods lists stress ratios in ascending order.

7.2.2 Test Procedures for Subgrade Materials

Specimen preparation

Due to the grain size of these materials being much smaller that that of granular materials, the minimum recommended specimen size is 50 mm diameter and 100 mm high. However, such small specimens make it difficult to place instrumentation on and it is recommended, in the CEN methods, that a diameter of 75 mm be used.

Specimen Density and Moisture Content

Again the specimen compaction technique may be either static or dynamic. The CEN specification simply requires that a specific density, at a particular moisture content, is attained. Although some guidance is given in that a method of compacting specimens at optimum moisture content is recommended and specimens are then dried in an oven to a predetermined weight (moisture content), after which they are given time to allow the moisture to reach equilibrium within the specimen.

It is generally acknowledged that routine modulus determination tests should be conducted in the undrained condition, without pore pressure measurement. However the moisture content may be increased, under controlled conditions by adopting the back pressure saturation technique. This technique allows design moisture contents, other than *in-situ*, for undisturbed specimens to be tested.

Strain Measurements and Applied Specimen Load

The recommendations made for the granular materials are the same for the subgrade soils with respect to the measurement of strains and application of loads.

Conditioning of the specimen is required by the CEN method of 80,000 load applications, at defined axial and radial stresses.

Determination of Resilient Behaviour

All methods recommend that the determination of the resilient modulus must be made via a number of different loading stresses following preconditioning. Again, AASHTO and ARRB state that these should proceed in a descending order of stress ratio whereas the CEN methods lists stress ratios in ascending order.

7.3 PHASE 1 - FIRST INTER-LABORATORY COMPARISON

The first test procedure (Test Programme I) was compiled from the experience of the Science Project's participants within their own laboratories during the early part of the project. These comparative tests were conducted on both unbound granular base materials and subgrade soils so that all the apparatus, from all four laboratories would be included, as follows:

Test Programme I on Subgrade Soils

Fontainebleau Sand	(SFB)	Tested Dry
London Clay	(LOC)	Partially Saturated

Test Programme I on Unbound Granular Materials

Soft Limestone	(CCD)	Partially Saturated
Hard Limestone	(CCT)	Partially Saturated
Microgranite	(MIG)	Partially Saturated

Table 7-1 Test Procedure I for the Subgrade Soils

Summary of the First Test Programme - Subgrade Soils

Aim: To compare the potential test methods and the triaxial equipment as used by the four participating laboratories, and to observe the behaviour of various materials under repeated loading.

Material:
Fountainbleau Sand (SFB) Undrained Failure Line q_f = 1.71 p + 35.6 kPa at ω = 0 %
Seine et Marne Silt (LIM) Undrained Failure Line q_f = 0.6 p + 108 kPa at ω = 20 %
London Clay (LOC) Undrained Failure Line q_f = 86 kPa at ω = 36 %

Compaction Methods:

Material	Compact. Method	Proctor Density	Dry Density (kg/m³)	ω (%)
SFB	Vib.Table	100%	1600	0.0
LIM	Tamping	95%	1760	17.2
LOC	Tamping	95%	1515	36.0

Laboratory:

Lab.	Specimen Size	CCP/VCP
LNEC	144 x 76 mm	VCP
UNOT	144 x 76 mm	VCP
LRCF	140 x 70 mm	VCP
DUT	210 x 102 mm	CCP

Loading: Resilient - Haversine wave form, frequency 1 second loading 1 second rest.
Conditioning/ Permanent - Simple sine wave at 5 Hz.

Test: A. Conditioning - (200 cycles)

σ_3 = 15 kPa (CCP) q_r/p_r = 3.0
q max = 0.5 q failure

B. Resilient Deformation - (50 cycles/ stress path)

Never exceed q max = 0.5 q failure

1 σ_3 min = 15; 30; 60 kPa (CCP) q_r/p_r = 3.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 5; 15; 30; 45; 60; 75; 90 kPa

2 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 2.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 5; 15; 30; 45; 60; 75; 90 kPa

3 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 1.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 5; 15; 30; 45; 60; 75; 90 kPa

4 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 1.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 5; 15; 30; 45; 60; 75; 90 kPa

5 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 0.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 5; 15; 30; 45; 60; 75; 90 kPa

C. Isotropic Loading - (50 cycles/ stress path)

At each confining pressure the deviator stress is zero and constant

1 σ_3 min = 15 kPa σ_3 max = 20; 30 kPa (VCP) q_r/p_r = 0.0

2 σ_3 min = 30 kPa σ_3 max = 35; 45; 60 kPa (VCP) q_r/p_r = 0.0

3 σ_3 min = 60 kPa σ_3 max = 65; 75; 90; 120 kPa (VCP) q_r/p_r = 0.0

D. Permanent Deformation - (100 000 cycles)

σ_3 min = 60 kPa σ_3 max = 60 kPa (CCP) q_r/p_r = 3.0

Where: CCP - Constant Confining Pressure
VCP - Variable Confining Pressure
Vib.Ham. - Vibrating Hammer
Vib.Tab. - Vibrating Table
Vib.Comp. Vibro Compresion Method

Table 7-2 Test Procedure I for the Unbound Granular Materials

Summary of the First Test Programme - Unbound Granular Materials

Aim: To compare the potential test methods and the triaxial equipment as used by the four participating laboratories, and to observe the behaviour of various materials under repeated loading.

Material:
Hard Limestone (CCD) — Undrained Failure Line q_f = 1.6 p + 30 kPa at ω = 3.3 %
Soft Limestone (CCT) — Undrained Failure Line q_f = 1.7p + 136 kPa at ω = 3.5 %
Microgranite (MIG) — Undrained Failure Line q_f = 2.28 p + 61 kPa at ω = 3.3 %

Compaction Methods:

Material	Modified Proctor	Dry Density (kg/m³)	ω (%)
CCD	98%	2370	3.3
CCT	98%	2250	3.5
MIG	98%	2150	3.3

Laboratory:

Lab.	Specimen Size	CCP/VCP	Compact. Method
LNEC	600 x 300 mm	CCP	Vib.Ham.
UNOT	300 x 150 mm	VCP	Vib.Tab.
LRSB	320 x 160 mm	VCP	Vib.Comp.
DUT	800 x 400 mm	CCP	Tamping

Loading: Resilient - Haversine wave form, frequency 1 second loading 1 second rest.
Conditioning/ Permanent - Simple sine wave at 5 Hz.

Test: A. Conditioning - (100 000 cycles)

σ_3 = 15 kPa (CCP) q_r/p_r = 3.0
q max = 0.5 q failure

B. Resilient Deformation - (50 cycles/ stress path)

Never exceed q max = 0.5 q failure

1 σ_3 min = 15; 30; 60 kPa (CCP) q_r/p_r = 3.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60; 75; 100; 150; 200; 300; 400 kPa

2 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 2.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60; 75; 100; 150; 200; 300; 400 kPa

3 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 1.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60; 75; 100; 150; 200; 300; 400 kPa

4 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 1.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60; 75; 100; 150; 200; 300; 400 kPa

5 σ_3 min = 15; 30; 60 kPa (VCP) q_r/p_r = 0.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60; 75; 100; 150; 200; 300; 400 kPa

C. Isotropic Loading - (50 cycles/ stress path)

At each confining pressure the deviator stress is zero and constant

1 σ_3 min = 15 kPa σ_3 max = 30; 45; 60 kPa

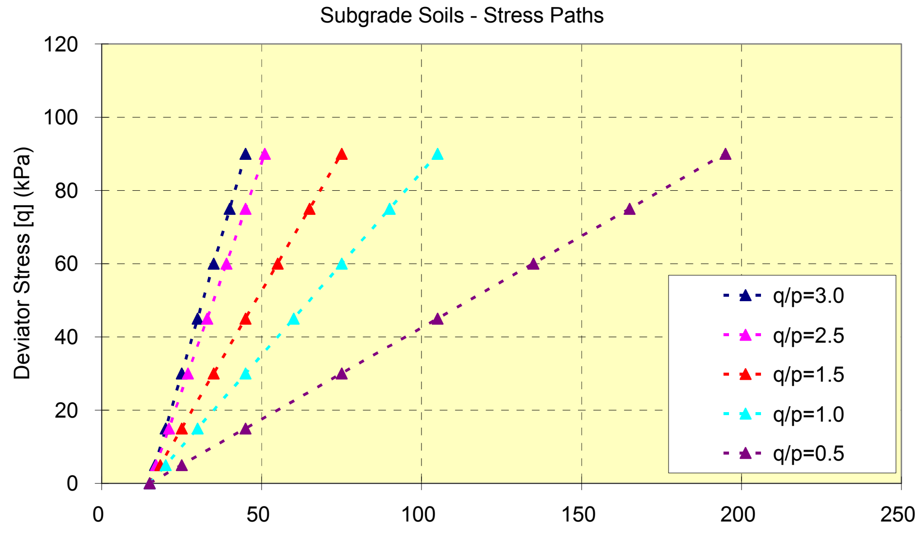
2 σ_3 min = 30 kPa σ_3 max = 45; 60; 90; 120 kPa

3 σ_3 min = 60 kPa σ_3 max = 75; 90; 120; 150; 180; 240 kPa

Where: CCP - Constant Confining Pressure; VCP - Variable Confining Pressure
Vib.Ham. - Vibrating Hammer; Vib.Tab. - Vibrating Table; Vib.Comp. Vibro Compresion Method

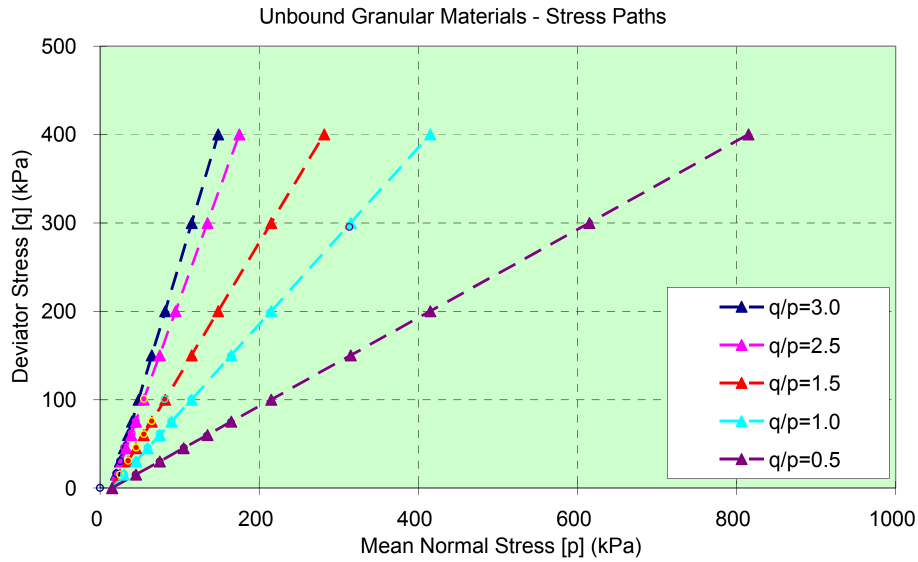
A summary of the two test procedures is shown in Table 7-1 and Table 7-2, the detailed test method and procedure is contained in Appendix D. The intended stress paths to be applied to the specimens (subject to the specimens being strong enough such they continue to exhibit largely resilient behaviour under the stress path) are shown graphically in Figure 7-1 and Figure 7-2, in p-q space.

Figure 7-1 Graphic Representation of Intended Stress Paths for Test Procedure I (Subgrade Soils)



Note: The triangles represent the ends of particular stress paths, which all start at q=0

Figure 7-2 Graphic Representation of Intended Stress Paths for Test Procedure I (Unbound Granular Materials)



Note: The triangles represent the ends of particular stress paths, which all start at q=0

It must be noted that it may not be possible to apply all of these stress paths due to the restriction that the maximum deviator stress applied should not exceed 50% of the deviator stress at failure as stated in the test procedure tables (Table 7-1 and Table 7-2).

These were determined for each material by conducting undrained standard triaxial tests to failure at different confining pressures. Strictly speaking these failure lines should be horizontal for these undrained conditions. However, since the subgrade soil is in an unsaturated state there is some effective stress change under loading due to movement of water within the material. For the granular material the angle of the failure line is mainly due to friction between particles under loading and some cohesion or suction within the material.

Since specimen failure will result if the stress paths are allowed to cross the failure lines those stress paths that cross the failure line were not applied to the specimens. It should be noted that the apparatus that cannot vary the confining pressure can only apply stress paths where q/p = 3.0, i.e. the large apparatus for testing unbound granular materials at LNEC and DUT, as described in an earlier chapter.

Each laboratory attempted to follow the test procedure as closely as possible. Restrictions were encountered due to the existing specimen preparation methods at the laboratories and it was found to be difficult for the laboratories to get the materials to exactly the specified moisture content and density. Since only one specimen was stipulated in the test procedure for each laboratory no checks within the particular laboratory were possible. It was noted that the specimen manufacture methods, including compaction methods, for the laboratories varied widely.

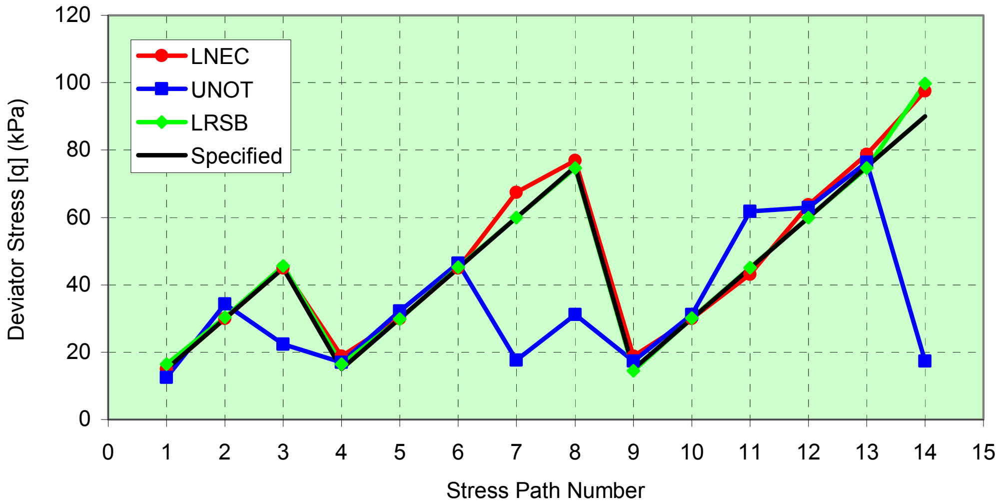
As stated some laboratories were unable to perform the repeated variation of the confining pressure as required by the test procedure and it was recognised that a more complex test regime was required with the stress path q/p = 3.0 so that a substantial number of test results could be obtained.

Importantly it was noted that the load applications applied by the various different apparatus were not particularly accurate. The measured loads varied quite considerably from the specified load regime as is shown in the example in Figure 7-3, where the UNOT applied deviator stress can clearly be seen to vary from that specified. The fourteen stress paths applied (graphically shown in Figure 7-3) are those under Test Programme I for a Hard Limestone (CCT) that do not cross the 50% Failure Line and are listed in Table 7-3.

Table 7-3 Stress Paths Test Programme I for a Hard Limestone (CCT)

Stress Path No.	Start Point (kPa)		End Point (kPa)							
			Specified		LNEC		UNOT		LRSB	
	p_1	q_1	p_2	q_2	p_2	q_2	p_2	q_2	p_2	q_2
1	15	0	20	15	21.0	15.0	19.2	12.5	20.6	16.4
2	15	0	25	30	26.0	30.0	26.5	34.4	25.2	30.4
3	15	0	30	45	31.0	45.0	32.5	22.4	30.4	45.7
4	30	0	35	15	35.6	18.7	35.7	17.0	35.8	16.4
5	30	0	40	30	39.3	30.0	40.7	32.2	40.2	29.8
6	30	0	45	45	44.3	45.0	45.5	46.4	45.4	45.2
7	30	0	50	60	51.8	67.5	40.9	17.6	50.2	59.9
8	30	0	55	75	55.0	76.9	50.4	31.2	55.1	74.6
9	60	0	65	15	67.6	18.7	65.8	17.4	65.3	14.5
10	60	0	70	30	71.3	30.0	70.4	31.1	70.4	30.1
11	60	0	75	45	75.7	43.1	80.6	61.8	75.0	45.1
12	60	0	80	60	82.6	63.7	81.0	62.9	80.0	60.0
13	60	0	85	75	87.6	78.7	85.5	76.4	84.9	74.7
14	60	0	90	90	93.8	97.5	70.8	17.4	93.4	99.7

Figure 7-3 Comparison of the Deviator Stresses Applied compared to that Specified for Different Laboratories for Hard Limestone



The results of the tests on both the subgrade soils and the unbound base materials were very scattered giving rise to a wide range of results. This is clearly illustrated in Figure 7-4 and Figure 7-5, which show the actual strains measured in three apparatus. Clearly, there is a great deal of discrepancy between the results; some laboratories measuring values consistently higher than others, and some laboratories showing greater scatter than others in their data.

The combination of inaccurate load applications and differing strain measurements is partly solved by normalising the strains with stress. For this example the strains have been divided by the deviator stresses and the results are shown in Figure 7-6. This figure shows that the strains measured at low stresses (early stress paths) are more scattered than those with larger load applications. However, what it does not show well is that the difference between normalised strains is equally poor for all stress paths, this is shown in Table 7-4. The percentage difference for the axial strain is only better that 50% in five of the thirteen stress paths and only a single stress path for the radial strain measurements. The average difference for the normalised axial strain is 53% and that for the normalised radial strain 68% implying that the radial strain measurement is less accurate than the axial strain measurement for the apparatus.

Figure 7-4 Comparison of the Axial Strain Measured at Different Laboratories for a Specimen of Hard Limestone

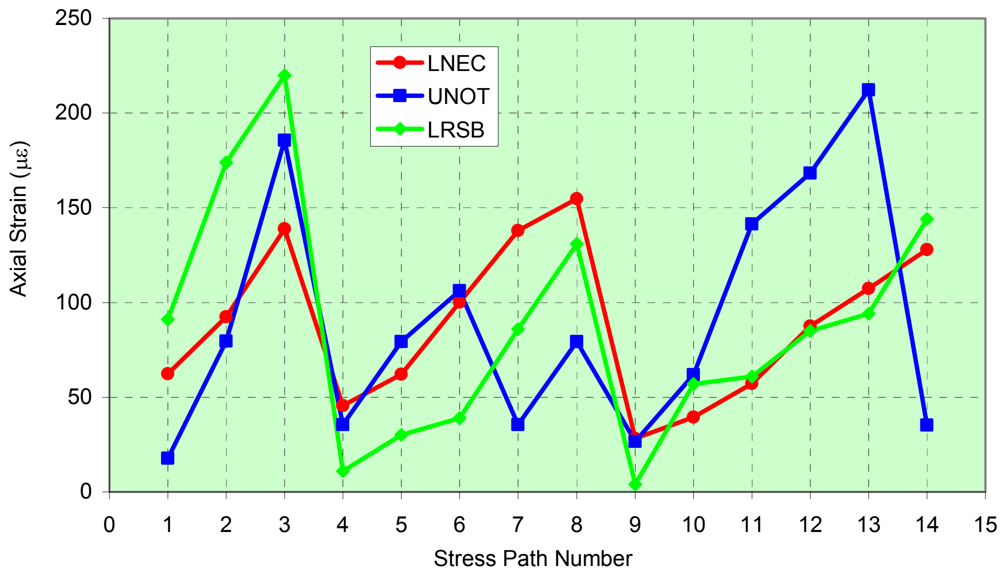


Figure 7-5 Comparison of the Radial Strain Measured at Different Laboratories for a Specimen of Hard Limestone

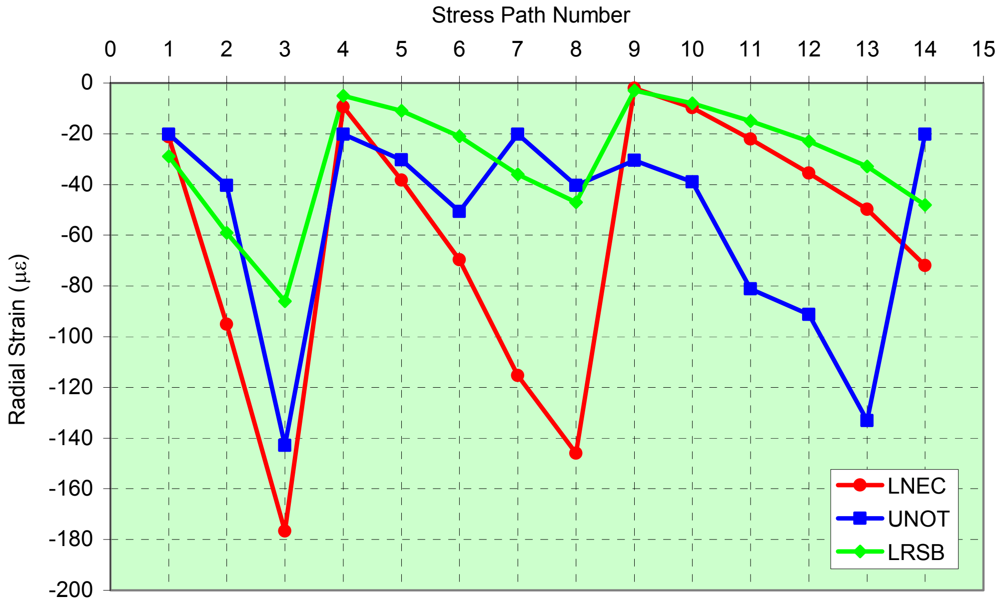


Figure 7-6 Comparative Strain Reading Normalised with Deviator Stress Paths for a Specimen of Hard Limestone

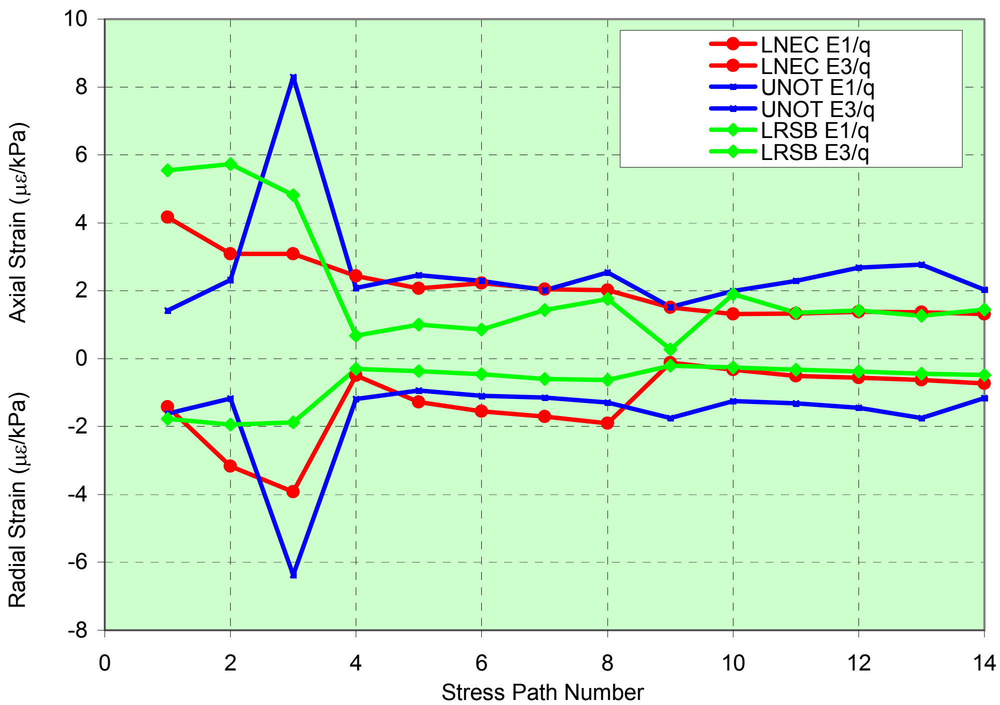


Table 7-4 The Range of Normalised Axial and Radial Strain measured at Different Laboratories for Hard Limestone

Stress Path No.	Normalised Axial Strain ε_1/q ($\mu\varepsilon$/kPa)			Normalised Radial Strain ε_3/q ($\mu\varepsilon$/kPa)		
	Min	Max	Diff (%)	Min	Max	Diff (%)
1	1.4	5.5	74%	-1.8	-1.4	20%
2	2.3	5.7	60%	-3.2	-1.2	63%
3	3.1	8.3	63%	-6.4	-1.9	70%
4	0.7	2.4	72%	-1.2	-0.3	75%
5	1.0	2.5	59%	-1.3	-0.4	71%
6	0.9	2.3	62%	-1.5	-0.5	70%
7	1.4	2.0	30%	-1.7	-0.6	65%
8	1.8	2.5	31%	-1.9	-0.6	67%
9	0.3	1.5	82%	-1.8	-0.1	93%
10	1.3	2.0	34%	-1.3	-0.3	79%
11	1.3	2.3	42%	-1.3	-0.3	75%
12	1.4	2.7	49%	-1.5	-0.4	74%
13	1.3	2.8	55%	-1.7	-0.4	75%
14	1.3	2.0	35%	-1.2	-0.5	59%

During this first test procedure a conditioning phase of only 200 load cycles was specified for the subgrade soil samples as opposed to 100,000 for the granular base materials. The specimen conditioning allows the large permanent strains, which occur during the first few thousand cycles, to take place, after which the specimen becomes almost entirely elastic. However, during these tests, after the conditioning phase, permanent deformations in the subgrade soil samples were still found to be occurring. This indicated that the conditioning phase was not sufficient to stabilise the permanent strains. It was concluded that a second inter-laboratory comparison (Phase 2) was necessary with a modified test procedure, primarily:

- Three specimens were to be tested for each material to allow comparison within a single laboratory; and,
- A larger number of load cycles were to be applied during the conditioning stage of the tests on subgrade soils to ensure that permanent deformation had stabilised.

Further, based on these results, it was also concluded that even if the composition of the materials was exactly the same there is a high likelihood that the degree of compaction and moisture content would differ due to differences in the specimen manufacturing methods employed by the different laboratories. Therefore, to obtain a better insight into the real differences in measuring systems, a test programme using an artificial specimen with known properties would be set up (Phase 3). All of the results for this test programme (Phase 1) are contained in Appendix F.1.

7.4 PHASE 2 - SECOND INTER-LABORATORY COMPARISON

Test Programme II was greatly simplified considering the problems encountered thus far. Only one subgrade soil (London Clay) and one unbound granular material (Microgranite) were used. These were conditioned and packaged from a single source, that is base material from Nottingham and subgrade soil from Lisbon, to prevent contamination errors of the materials. The material characteristics under which these two materials were tested are shown in Table 7-5. Detailed summaries of the test procedures are contained in Appendix D.2 and are summarised in Table 7-6 and Table 7-7.

Table 7-5 Materials Characteristics as Tested in Phase 2

Soil Type	Maximum Particle Size (mm)	w_{OMC} (%)	ρ_{OMC} (kg/m³)	w_{Test} (%)	ρ_{Test} (kg/m³)
UGM (Microgranite)	31.5	5.3	2,180	3.3	2,140
Soil (London Clay)	-	36.0	1,370	36.0	1,370

Notes:
1. Proctor compaction for soil, modified Proctor for UGM.
2. Compaction methods of the laboratories varied widely.
3. A test dry density of only ρ_{Test} = 1,230 kg/m³ was achieved at UNOT for London Clay.

A detailed comparison was made in Progress Report No.1 (1990) of each laboratory's apparatus and specimen manufacture procedures and this test procedure took into consideration the differing preparation of specimens. It furthermore, specified the loading criteria in accordance with the potential of each laboratory's equipment.

Table 7-6 Test Procedure II for the Subgrade Soils

Summary of the Second Test Programme - Subgrade Soils

Aim: To Compare measured deformations on different Triaxial equipment using identical material and procedure.

Material: London Clay (LOC)
Compaction energy of standard normal proctor
Moisture Content ω = 36.0%
Undrained Failure Line q_f = 86 kPa at ω = 36.0% (undrained test)

Compaction Methods:

Laboratory	Method (MC %)	Specimen Size	CCP/VCP
LNEC	Tamping	144 x 76 mm	VCP
LRCF	Tamping	140 x 70 mm	VCP
UNOT	Tamping	144 x 76 mm	VCP
DUT	Tamping	200 x 100 mm	CCP

Loading: Resilient - Haversine wave form, frequency 1 second loading 1 second rest.
Permanent - Simple sine wave form, frequency 5 Hz.

Test: A. Isotropic Loading I - (10 cycles/ stress path)

The cell pressure is cycled between 0 kPa and 15; 30; 45 kPa.

B. Resilient Deformation I - (50 cycles/ stress path)

σ_3 min = 15 kPa σ_3 max = 15 kPa (CCP) q_r/p_r = 3.0
At this confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30 kPa

C. Resilient Deformation II - (50 cycles/ stress path)

σ_3 min = 30 kPa σ_3 max = 30 kPa (CCP) q_r/p_r = 3.0
At this confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45 kPa

D. Resilient Deformation III - (50 cycles/ stress path)

σ_3 min = 60 kPa σ_3 max = 60 kPa (CCP) q_r/p_r = 3.0
At this confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60 kPa

E. Isotropic Loading II - (10 cycles/ stress path)

The cell pressure is cycled between 0 kPa and 15; 30; 45; 60; 75 kPa.

F. Permanent Deformation - (100 000 cycles)

σ_3 min = 60 kPa σ_3 max = 60 kPa (CCP) q_r/p_r = 3.0
q min = 0 kPa q max = 80 kPa.

Where: CCP - Constant Confining Pressure
VCP - Variable Confining Pressure

Table 7-7 Test Procedure II for the Unbound Granular Materials

Summary of the Second Test Programme - Unbound Granular Materials

Aim: To Compare measured deformations on different Triaxial equipment using identical material and procedure.

Material: Microgranite (MIG)

Test Dry Density = 2140 kg/m^3 and Moisture Content ω = 3.3 %

Undrained Failure Line q_f = 2.28 p + 61 kPa at ω = 3.3%

Compaction Methods:

Laboratory	Method (MC %)	Specimen Size	CCP/VCP
LNEC	Vib.Hammer	600 x 300 mm	CCP
LRSB	Vib.Comp.	320 x 160 mm	VCP
UNOT	Vib.Table	300 x 150 mm	VCP
DUT	Tamping	800 x 400 mm	CCP

Loading Haversine wave form, 1 second load - 1 second rest.

Test: A. Isotropic Loading I - (5 cycles/ stress path)

The cell pressure is cycled between 0 kPa and 50 kPa for five cycles and the deflections measured.

B. Permanent Deformation - (100 000 cycles)

1 σ_3 min = 50 kPa σ_3 max = 50 kPa (CCP) Stress ratio (q_r/p_r) = 3.0
q min = 0 kPa q max = 50 kPa.

C. Isotropic Loading II - (5 cycles/ stress path) - as above

D. Resilient Deformation I - (100 cycles/ stress path)

1 σ_3 min = 15 kPa σ_3 max = 15 kPa (CCP) q_r/p_r = 3.0
At this confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 15; 30; 45; 60 kPa

2 σ_3 min = 30 kPa σ_3 max = 30 kPa (CCP) q_r/p_r = 3.0
At this confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 25; 50; 75; 100 kPa

3 σ_3 min = 45 kPa σ_3 max = 45 kPa (CCP) q_r/p_r = 3.0
At this confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 30; 60; 90; 120 kPa

E. Isotropic Loading III - (5 cycles/ stress path) - as above

F. Resilient Deformation II - (100 cycles/ stress path)

1 σ_3 min = 15 kPa σ_3 max = 55; 95; 135 kPa (VCP) q_r/p_r = 1.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 60; 120; 180 kPa

2 σ_3 min = 30 kPa σ_3 max = 70; 110; 150 kPa (VCP) q_r/p_r = 1.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 60; 120; 180 kPa

3 σ_3 min = 45 kPa σ_3 max = 85; 125; 165 kPa (VCP) q_r/p_r = 1.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 60; 120; 180 kPa

G. Isotropic Loading IV - (5 cycles/ stress path) - as above

Where: CCP - Constant Confining Pressure; VCP - Variable Confining Pressure; Vib.Ham. - Vibrating Hammer; Vib.Tab. - Vibrating Table; Vib.Comp. Vibro Compresion Method

As a consequence of this more effort was made in obtaining specimens, at the various laboratories, that were at the same densities and moisture condition. Some investigation into the variation of results, as a function of the methods of compaction of base material was conducted during this phase. The specified compaction methods are shown in Table 7-8.

Table 7-8 Compaction Methods Specified

Method	Laboratory	Specimen Size
Vibrating Hammer	LNEC	600 x 300 mm
Vibrating Compression Apparatus	LRSB	320 x 160 mm
Vibrating Table	UNOT	300 x 150 mm
Tamping	DUT	800 x 400 mm

Each of the laboratories had developed various methods of specimen manufacture and compaction and it was not possible to change this easily. The time and cost of developing and constructing a new compaction apparatus was considered to be too great. However, if a standard test procedure is to be formulated, consideration must be given to a standard method of compaction, which is relatively easy, quick and effective.

7.4.1 Permanent Strain Behaviour

The permanent strain on subgrade soils during conditioning was not measured. This was because the conditioning is meant to simply bed the specimen against the apparatus platens and to ensure that the instrumentation is functioning in order that the specimens are ready for the resilient phase of the test rather than measure the permanent deformation under loading. Once the resilient tests are completed then a higher loading magnitude is applied to the specimens and the permanent deformation measured.

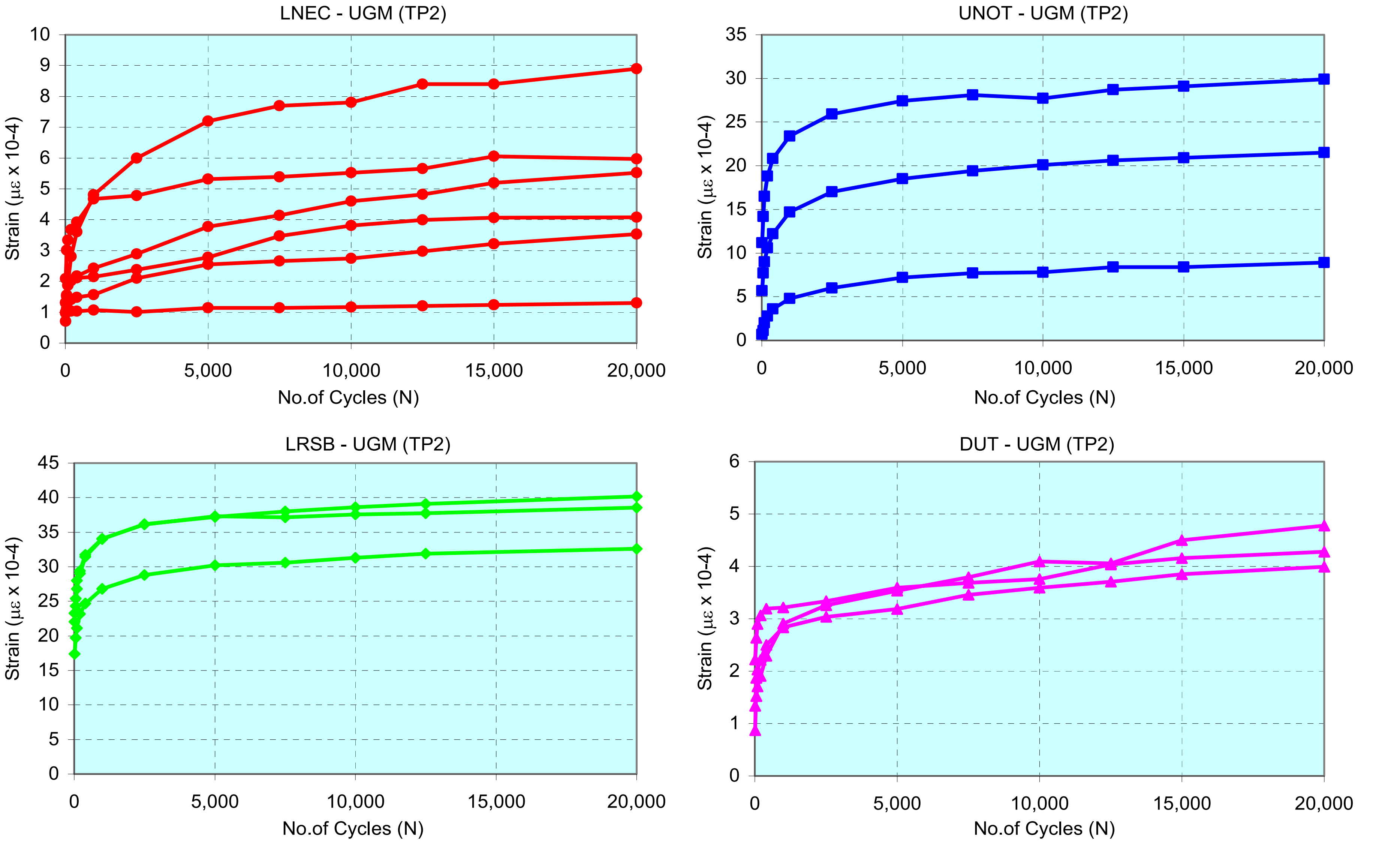
The axial permanent strains measured on the unbound granular specimens by the four laboratories were found to vary somewhat as is clearly shown in Figure 7-7. The mean values range from 3.6 x 10^{-4} $\mu\varepsilon$ at LNEC to 37.0 x 10^{-4} $\mu\varepsilon$ at LRSB, as shown in Table 7-9, a factor of approximately 10, which is an unacceptable difference. It was concluded that this range is probably caused by differences in the compaction method used by the laboratories. Methods of compaction which induce high levels of shear, such as the vibrating hammer (LNEC) and manual tamping under cyclic preloading (DUT) result in lower permanent strains than the methods where the compaction is full face and tend not to induce such high levels of shear, such as the vibrocompression apparatus (LRSB) and vibrating table and full face static load (UNOT).

Table 7-9 Comparison of the Permanent Axial Strain for Unbound Granular Specimens

Tests Conducted	Strain Values after Specimen Conditioning [20,000 cycles] at Various Laboratories ($\mu\varepsilon$ x 10^{-4})			
	LNEC	UNOT	LRSB	DUT
Specimen 1	4.1	8.9	32.6	4.3
Specimen 2	1.3	29.9	38.6	3.9
Specimen 3	5.5	21.5	40.2	4.9
Specimen 4	3.5			
Mean	3.6	20.1	37.1	4.4
Standard Deviation	1.7	10.6	4.0	0.5
Coefficient of Variation	49%	53%	11%	12%

The mean value, from the results of the four laboratories is, 16 $\mu\varepsilon$, the standard deviation is 16 $\mu\varepsilon$. This is very poor since it is the same as the mean value, the Coefficient of Variation is 97%, which, too is very poor. During these conditioning tests, it was observed that the permanent strains measured per load cycle decreased significantly from the start of the load applications but stabilised after approximately 5,000 cycles. This confirms the importance of applying a cyclic conditioning before studying the resilient behaviour of a granular material.

Figure 7-7 Permanent Strains Measured in Different Apparatus while testing Microgranite



7.4.2 Resilient Strain Behaviour

Resilient tests were conducted on both granular base materials and subgrade soils at each of the four laboratories. Again different behaviour was found from one laboratory to another. The results, however, were better than those obtained during the earlier Test Programme I. An example of the test results for varying deviator stress for the subgrade soil and unbound granular material is shown in Figure 7-8 and Figure 7-9 respectively.

Unbound Granular Material

Resilient measurements, taken during the conditioning stage of the tests, on specimens of unbound granular material were conducted for the stress path of deviatoric stress cycled between 0 and 130 kPa with a constant confining pressure of 50 kPa. For the resilient axial strains, the agreement between the laboratories is much improved as shown in Table 7-10. UNOT obtained somewhat higher and more varied values, which was considered to be due to generally low and scattered dry densities attained during the manufacture of the specimens.

Table 7-10 Comparison of the Resilient Axial Strain for Unbound Granular Specimens (TP2)

Tests Conducted	Resilient Axial Strain ($\mu\varepsilon$) at p_1 = 50 kPa; q_r = 0 - 130 kPa			
	LNEC	UNOT	LRSB	DUT
Specimen 1	351	661	345	426
Specimen 2	309	568	378	350
Specimen 3	315	450	452	448
Specimen 4	392			
Mean	342	460	392	408
Standard Deviation	38	106	55	51
Coefficient of Variation	11%	19%	14%	13%

The mean value from the four laboratories is 401 $\mu\varepsilon$, the standard deviation is 49 $\mu\varepsilon$ and the Coefficient of Variation is 12%, which is a considerable improvement on the earlier test programme.

Figure 7-8 Resilient Strains Measured on Specimens of Subgrade Soil during Test Programme II

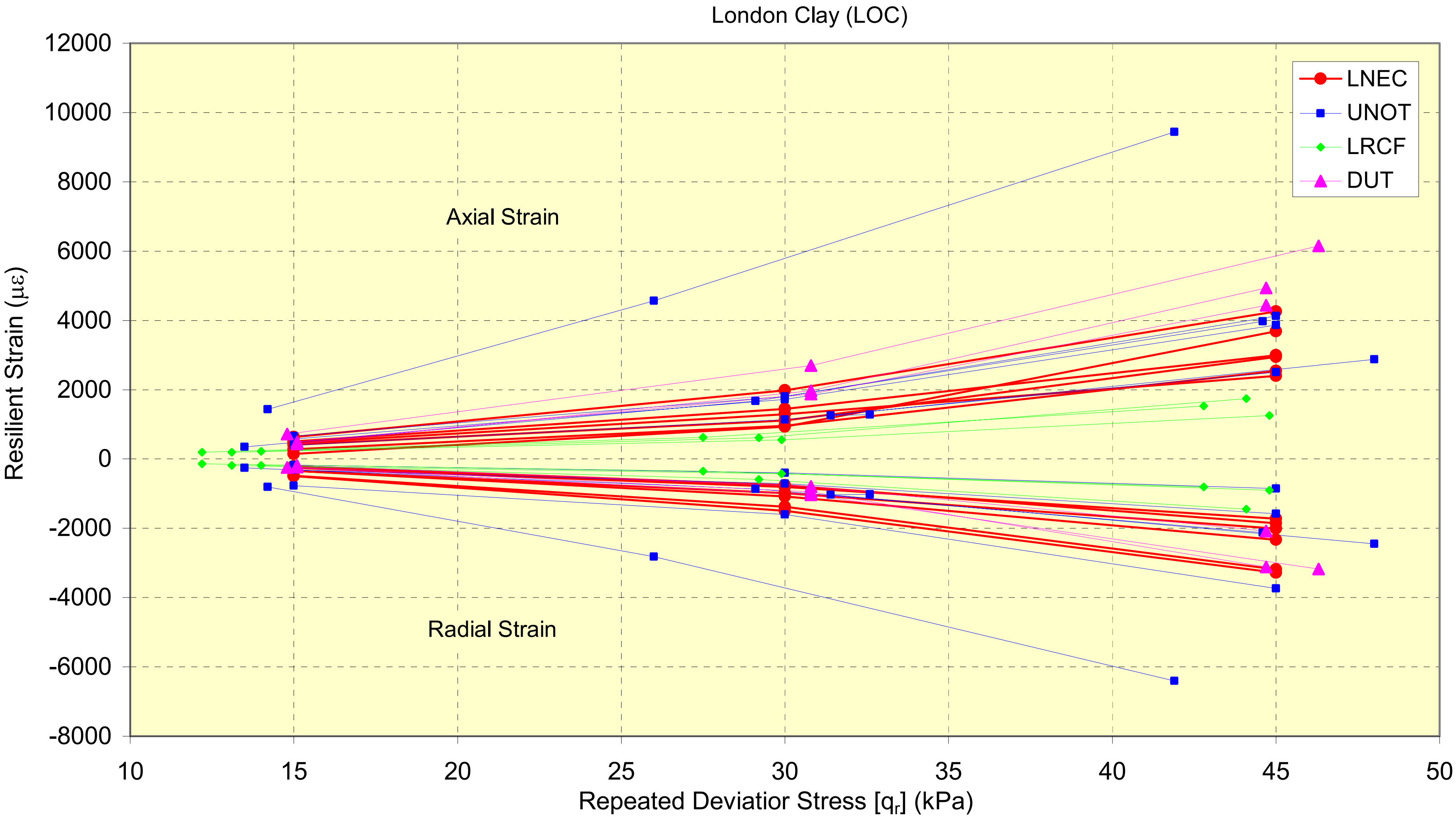
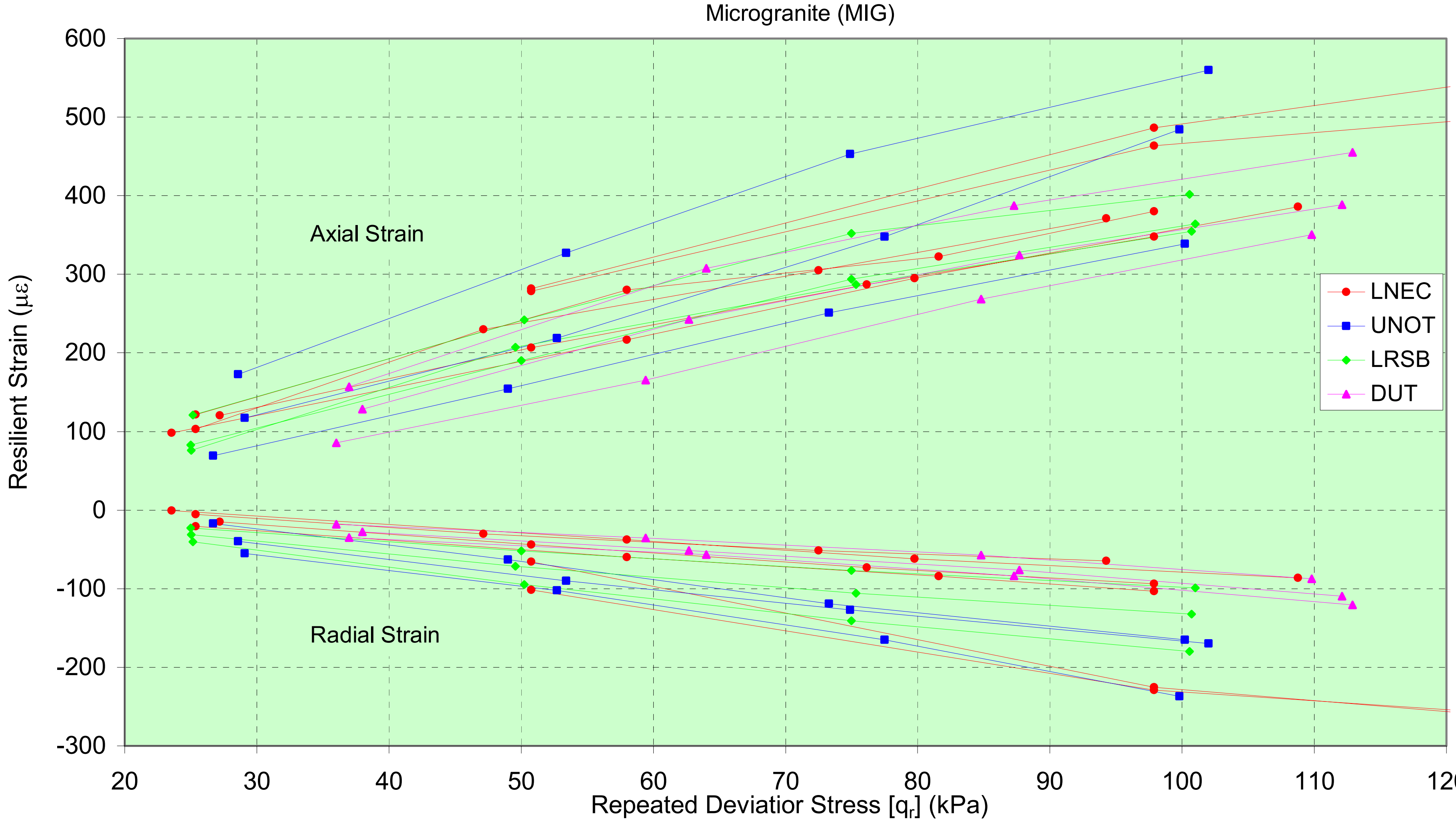


Figure 7-9 Resilient Strains Measured on Specimens of Unbound Granular Base during Test Programme II



The radial resilient strains shown in Table 7-11 are less satisfactory since there is a large variation between the results, with UNOT being about 4 times greater than LNEC.

Table 7-11 Comparison of the Resilient Radial Strain for Unbound Granular Specimens (TP2)

Tests Conducted	Resilient Radial Strain ($\mu\varepsilon$) at p_1 = 50 kPa; q_r = 0 - 130 kPa			
	LNEC	UNOT	LRSB	DUT
Specimen 1	-77	-198	-97	-131
Specimen 2	-37	-254	-135	-169
Specimen 3	-38	-218	-176	-138
Specimen 4	-72			
Mean	-56	-223	-136	-146
Standard Deviation	21	28	40	20
Coefficient of Variation	38%	13%	29%	14%

The mean value from the four laboratories is -140 $\mu\varepsilon$, the standard deviation is 68 $\mu\varepsilon$ and the Coefficient of Variation is 49%, which is a great deal poorer than the 12% of the axial resilient strain results above. It is thus surmised that there are greater errors in the radial measurement systems than the axial measuring systems, something that was shown during the earlier test programme as well.

For the resilient axial strains, relatively small differences were observed between the mean values from the four laboratories but the coefficient of variation (repeatability) was much greater. The best repeatability was obtained where large specimens were used (LNEC and DUT). In addition the best repeatability was obtained where less scattered dry densities were recorded (LNEC and LRSB), thus the method of compaction is important.

For the radial strains and for the Poisson's ratio (which is largely dependent on radial strains accuracy), the variation of the results was large at all the laboratories. These poor results show that there are difficulties associated with the measurement of radial strains in these triaxial tests. This can be explained by the fact that the radial strains measured were particularly small, for the LRSB specimens, for example. This corresponds to radial displacements lower than 16 $\mu\varepsilon$, which is near the limit of the capability of some instrumentation. Further, fixing of the radial instrumentation to the specimen poses difficulties and is a probable source of errors. The compaction of the material around the studs, which are embedded into the materials, or the effect of the membrane when the instrumentation is glued on to the membrane could have caused errors.

Subgrade Soils

Unfortunately, LNEC did not record the strains during the 100,000 cycle conditioning phase. UNOT and DUT recorded permanent axial strains of similar magnitude during the conditioning phase as shown in Table 7-12. LRCF recorded very small values, about 60 times smaller than the other laboratories, indicating some error. Larger differences were found between the mean values of the permanent radial strains obtained by the three laboratories. The variation of results from a single laboratory was found to be greater, with coefficients of variation of between 20 and 40%. Again, LRPC recorded extremely small strains in comparison with the other laboratories.

Table 7-12 Comparison of the Permanent Strains for Subgrade Soil Specimens (TP2)

Tests Conducted	**Strain Values after Specimen Conditioning [100,000 cycles] at Various Laboratories ($\mu\varepsilon$ x 10^{-4})**					
	Axial Strain			**Radial Strain**		
	UNOT	**LRCF**	**DUT**	**UNOT**	**LRCF**	**DUT**
Specimen 1	148	3.5	335	-31.5	-1.2	-212
Specimen 2	231	3.5	194	-105.0	-1.9	-109
Specimen 3	225	3.6	211	-85.3	-1.3	-138
Mean	201	3.5	247	-73.8	-1.5	-153
Standard Deviation	37.9	0.0	63.1	31.0	0.3	43.6
Coefficient of Variation	19%	1%	26%	42%	21%	29%

Table 7-13 and Table 7-14 show a comparison of the strains measured on the specimens of subgrade soils conducted at the four different laboratories as part of Phase 2. These results are for a fixed confining pressure of 30 kPa and varying deviator stresses as shown.

Table 7-13 Comparison of the Axial Strains for London Clay Specimens (TP2)

Tests Conducted	Resilient Axial Strain ($\mu\varepsilon$) at p_1 = 30 kPa; q_r = 0 - 45 kPa			
	LNEC	UNOT	LRCF	DUT
Mean	3136	3465	1514	5173
Standard Deviation	645	651	200	719
Coefficient of Variation	21%	19%	13%	14%

Table 7-14 Comparison of the Resilient Strains London Clay Specimens (TP2)

Tests Conducted	Resilient Radial Strain ($\mu\varepsilon$) at p_1 = 30 kPa; q_r = 0 - 45 kPa			
	LNEC	UNOT	LRCF	DUT
Mean	-2396	-2147	-1049	-2789
Standard Deviation	623	960	284	502
Coefficient of Variation	26%	45%	27%	18%

Two laboratories (LNEC and UNOT) obtained similar mean axial strain values. The axial strain values measured at DUT were largest, which is to be expected, because the axial strain is measured with a transducer placed outside the triaxial cell and this tends to overestimate the strain {Lashine (1971), Parr (1972), Barksdale (1972b)}. The reason why LRCF obtained results of approximately half those of LNEC and UNOT is unclear.

Within a single laboratory, the smallest variation, or scatter of axial strain results, was found at LRCF, with a coefficient of variation of 13%. Since this laboratory has greater experience in this type of testing it is considered that both the operators and the methods of testing were more competent than those at the other laboratories, who has less experience in testing, and therefore the repeatability of tests was better. The largest scatter was observed at LNEC. This is thought to be due to the fact that this laboratory was still developing their apparatus and test procedures so that greater operator errors were likely to occur.

For resilient radial strains, as shown in Table 7-14, it was noted that LNEC, UNOT and DUT all used proximity transducers for measuring radial strains and obtaining mean radial strain values of fair agreement. LRCF, however, obtained systematically about 2 times smaller values of radial strain than the other laboratories.

The scatter of the radial strain results is larger than those found for axial strains. DUT had the lowest variation, which might be due to their using larger specimens (100 mm in diameter) than the other laboratories. The poorest variation was found at UNOT, with coefficients of variation exceeding 40 %.

The resilient modulus has been calculated for each of the different deviator stress applications and the non-linearity is clear as shown in Table 7-15. These results generally follow the trend of the strains, except that the differences between the laboratories seem smaller, for example 33; 31 and 29 MPa obtained for resilient modulus at q_r = 15 kPa.

Table 7-15 Comparison of the Resilient Modulus for Subgrade Soil Specimens (TP2)

Tests Conducted	**Resilient Modulus (MPa) at p_1 = 30 kPa**											
	LNEC			**UNOT**			**LRCF**			**DUT**		
Deviator Stress (kPa)	0-15	0-30	0-45	0-15	0-30	0-45	0-15	0-30	0-45	0-15	0-30	0-45
Mean	33	25	14	31	21	14	65	49	30	29	15	9
Standard Deviation	8.5	7.5	2.1	7.1	4.3	3.0	2.1	4.2	4.4	5.9	2.2	1.0
Coefficient of Variation (%)	26	30	15	23	21	22	3	9	15	21	15	12

After assessing the results of Phase 1 the test procedure was modified and a series of tests was conducted by each of the four laboratories on similar materials (Phase 2 and again somewhat different results in measured strains were obtained. It was thus decided to exclude all possible influences related to natural materials and specimen manufacturing differences and to test an artificial specimen (Phase 3). All of the results for this test programme (Phase 2) are contained in Appendix F.2.

7.5 PHASE 3 - ROUND ROBIN TESTING ON THE ARTIFICIAL SPECIMEN

The material used for these tests was Polytetrafluoroethylene (PTFE), as described in the previous chapter. In order to eliminate the time dependent specimen response, under loading, the generated loading signal was applied as a static square signal as shown in Table 7-16.

Table 7-16 Loading Regime Applied to the Artificial Specimen

Test No.	Time (sec)	Confining Pressure σ_3 (kPa)	Deviator Stress q (kPa)
1	0 - 3600	250	0
	3600 - 7200	0	0
2	0 - 3600	250	375
	3600 - 7200	0	0
3	0 - 3600	100	600
	3600 - 7200	0	0

As the laboratories had equipment of different dimensions, the artificial specimen was progressively reduced in size, as it was tested in smaller capacity apparatus. Thus, it may be argued, there was some difference between the specimens as assessed at the different laboratories. Nevertheless, this difference is believed to be small when compared with that inherent in unbound granular materials and soils. A limitation on the initial size of the artificial specimen meant that it was not possible to include the large DUT and LNEC apparatuses in the comparisons. The tests were conducted in laboratories as listed in Table 7-17. This table also shows the specimen size (thus identifies the actual apparatus) and the temperature at which the test was conducted.

The instruments were affixed to the artificial specimen with, as near as possible, the same methods as for the real specimens. The principal limitation was that studs/ vanes could not be embedded, so the external elements of the fixing were screwed into holes tapped in the specimen. It is anticipated that this would have introduced some improvement in performance over that recorded with real specimens.

Table 7-17 The Apparatus and the Corresponding Specimen Size

Apparatus	Specimen Size [Height x Diameter] (mm)	Temperature (°C)
LRSB	320 x 160	18
UNOT1	320 x 160	24
DUT[1]	210 x 102	22
LNEC	144 x 76	21
UNOT2	144 x 76	24
LRCF[2]	140 x 70	18

Notes:
1. Small static deviator stress (6 kPa) constantly applied.
2. Stress levels applied are lower than stipulated.

Applied Loads (Stress)

There was some variation in the applied load from that stipulated as shown in Table 7-18, which shows the recorded stresses. The stresses applied to the specimen at LRCF were much lower than those stipulated, this being due to deficiencies in their apparatus. Consequently these results are not included in the arithmetic means. With the exception of the LRCF loading the applied loading was very uniform with coefficients of variation of less than 1%.

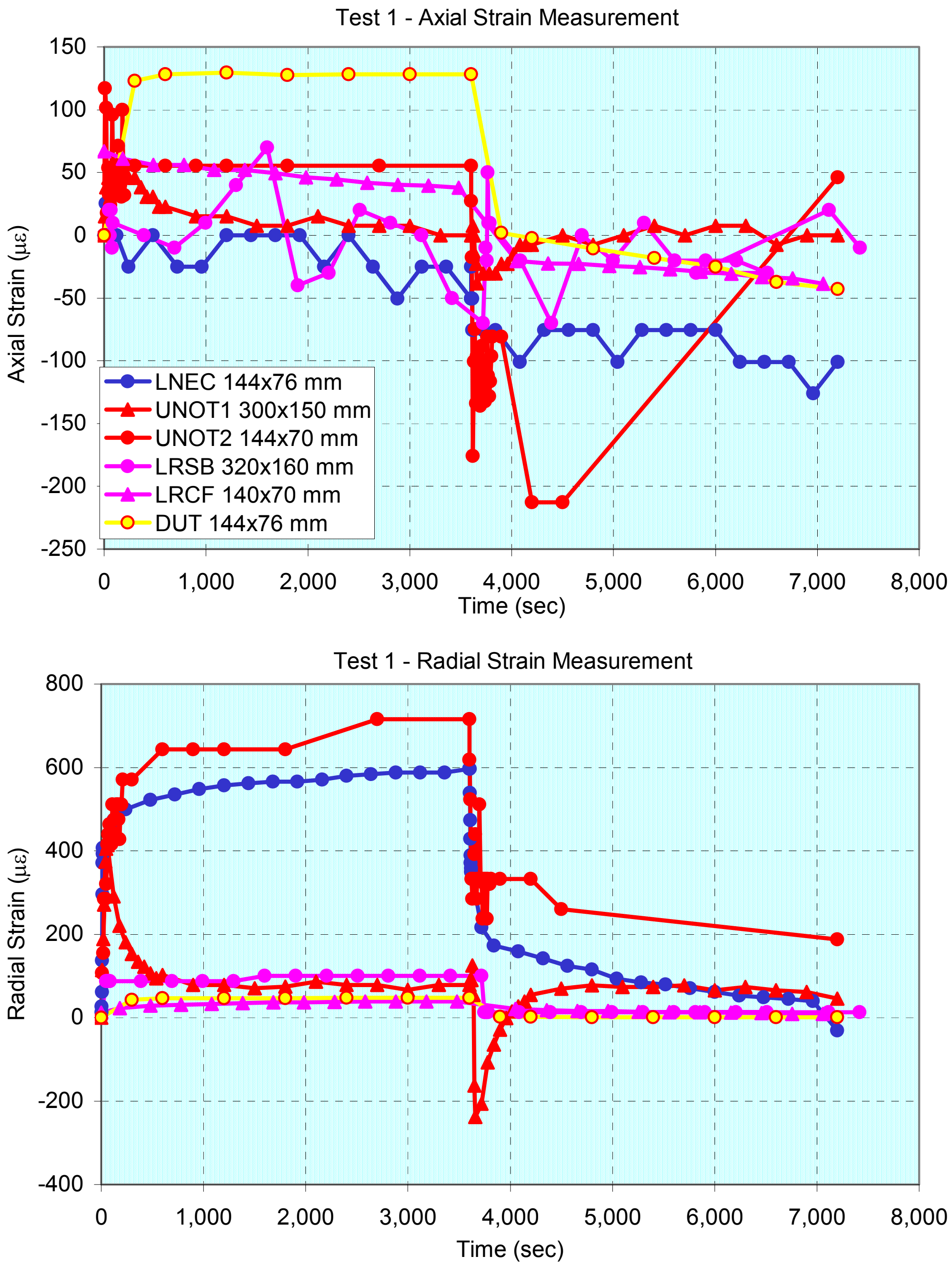
Table 7-18 Recorded Stresses Applied to the Artificial Specimen

Apparatus	Test 1		Test 2		Test 3	
	σ_{1r} (kPa)	σ_{3r} (kPa)	σ_{1r} (kPa)	σ_{3r} (kPa)	σ_{1r} (kPa)	σ_{3r} (kPa)
LNEC	250	250	643	249	710	98
UNOT1	248	248	618	246	701	100
UNOT2	250	250	625	250	700	100
LRSB	250	250	625	250	700	100
LRCF	*236*	*236*	*326*	*237*	*176*	*87*
DUT	250	250	626	250	686	100
Mean	250	250	627	249	699	100
Standard Deviation	1	1	9	2	9	1
Coefficient of Variation	0%	0%	1%	1%	1%	1%

Test 1 – Deformation (Strain) Measurement

The deformations measured on the specimens under this isotropic loading were very small, as is shown in Figure 7-10, due to small loading stresses. This test illustrates that the accuracy of the systems are limited. During these low stress levels the instrumentation was found to wander and since the strains were very small the wandering might have exceeded the actual strain. This is clearly demonstrated by the LNEC Axial data in Figure 7-10, which 'jumps' in multiples of 25 $\mu\varepsilon$.

Figure 7-10 Artificial Specimen Test 1



Thus there is an obvious error associated with the minimum reading that the instrumentation can measure. In general the instrumentation systems are only able to measure strains of greater than about 60 $\mu\varepsilon$ as shown in Table 7-19.

The radial strain measurements made by LNEC and UNOT2 are much higher that those of the other instruments. These apparatus are similar (both developed at Nottingham) and both use proximity transducers to measure the radial strain. The remaining instruments show similar magnitudes of strain measurement.

Complete results of the tests conducted during this experiment are contained in Appendix E.

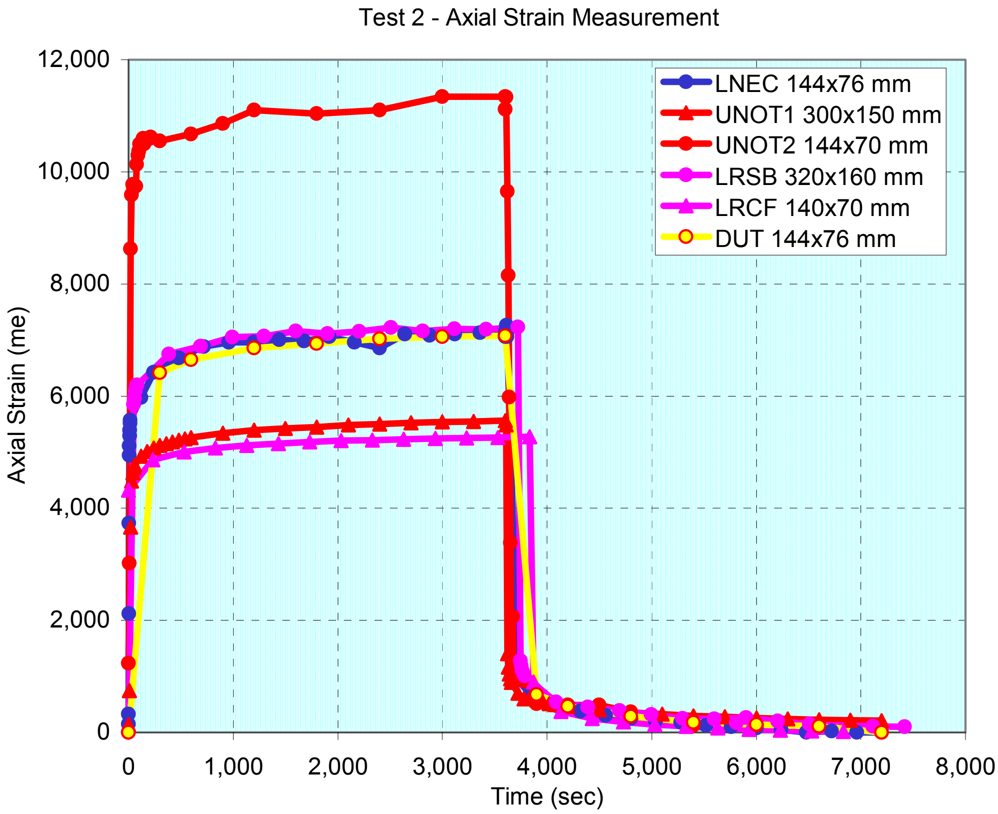
Table 7-19 The Average and Minimum Instrumentation Wandering

Apparatus	**Axial Strain ($\mu\varepsilon$)**		**Radial Strain ($\mu\varepsilon$)**	
	Average	**Minimum**	**Average**	**Minimum**
LNEC	25	25	22	4
UNOT1	9	8	26	4
UNOT2	36	3	60	12
LRSB	29	10	50	13
LRCF	3	2	3	1
DUT	20	1	16	1

<u>Test 2 and Test 3 – Deformation (Strain) Measurement</u>

Axial loading applied to the specimen in these two tests resulted in much larger strains being recorded. As shown in Figure 7-11 and Figure 7-12he axial load imposed by LRCF was considerably lower than that applied by the other laboratories, thus the strains are expected to be much less as is the case. UNOT yielded a much larger axial strain for their small apparatus and a lower axial strain for their large apparatus. Generally all of the radial strains coincide well. Again, however, some variation is seen in the UNOT results. The trends are the same for Test 2 and Test 3 for both the axial and radial strains. Table 7-20 contains a summary of the strains for each of the apparatus during these tests. The deviation from the arithmetic mean is shown in this table as a percentage in each case. These figures show that the deviation is quite large, as expected for LRCF, but also for UNOT.

Figure 7-11 Artificial Specimen Test 2



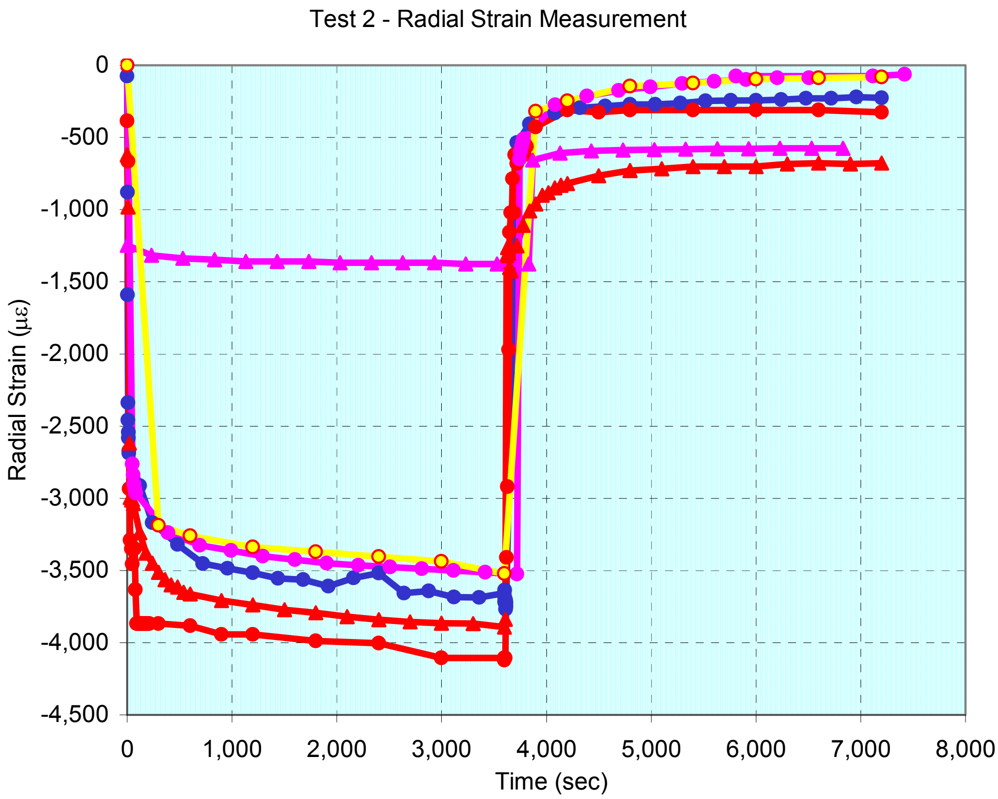
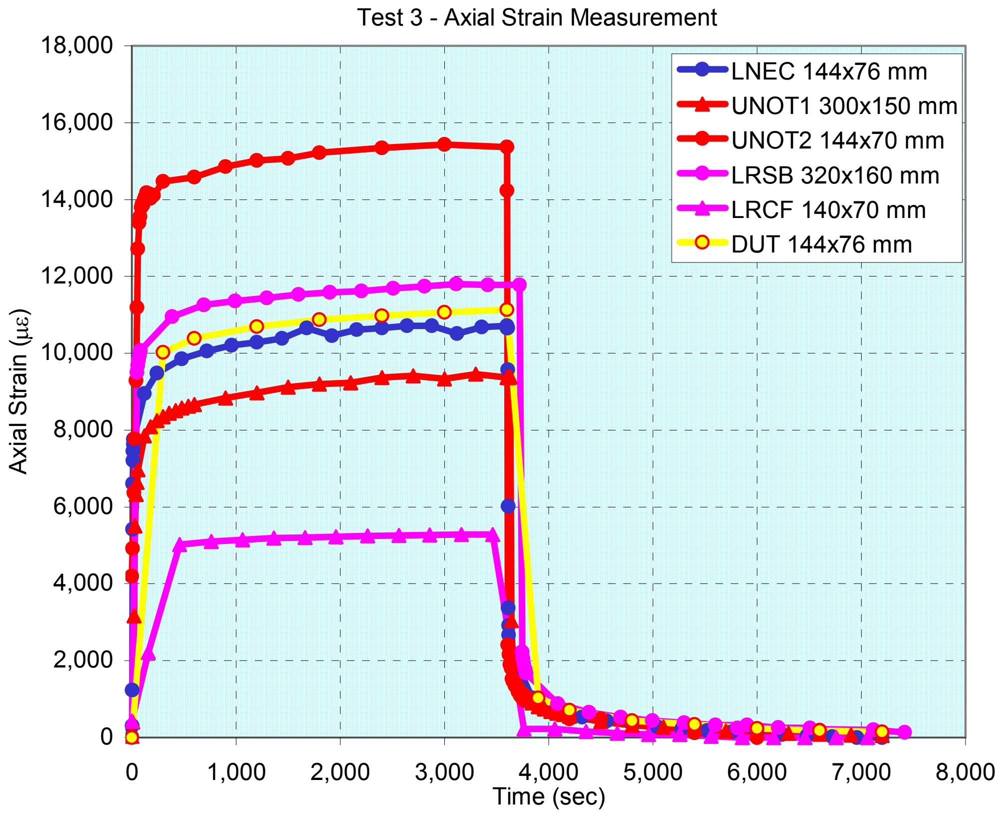


Figure 7-12 Artificial Specimen Test 3



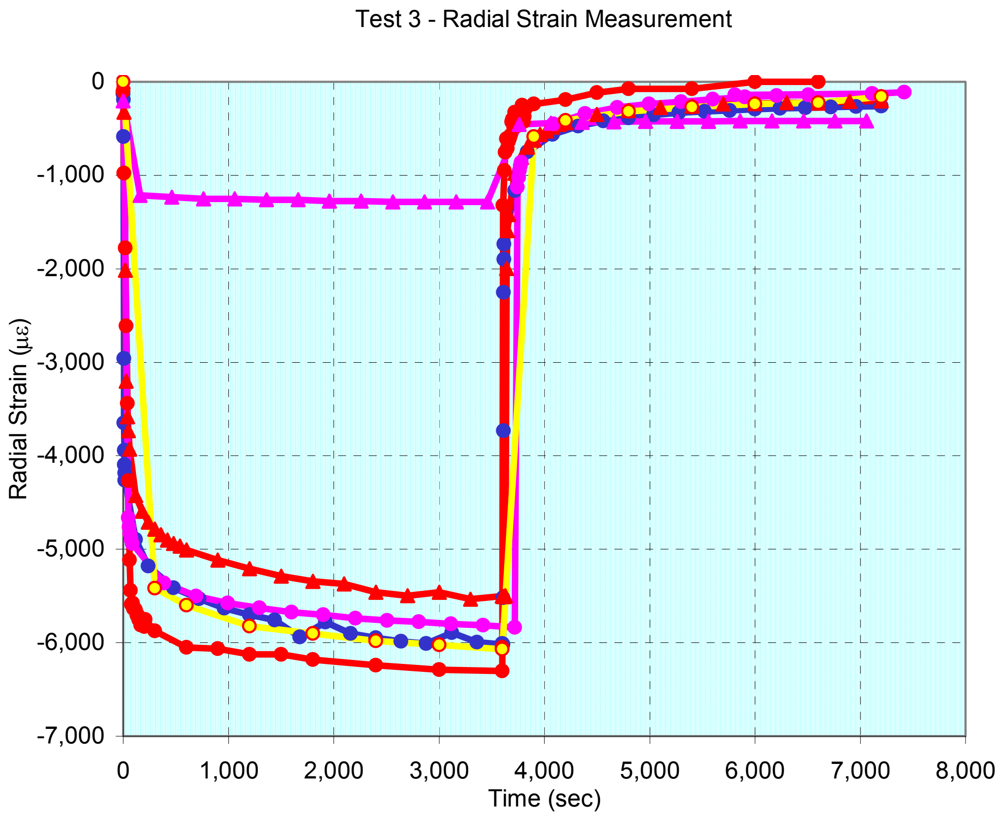


Table 7-20 Recorded Strains on the Artificial Specimen

Laboratory	Test 2				Test 3			
	Axial		Radial		Axial		Radial	
	(με)	d	(με)	d	(με)	d	(με)	d
LNEC	7081	6%	-3431	1%	10710	7%	-5756	0%
UNOT1	5357	29%	-3215	7%	9320	19%	-5296	8%
UNOT2	10978	46%	-3796	9%	14858	29%	-6049	5%
LRSB	7130	5%	-3463	0%	11640	1%	-5725	0%
LRCF	*5252*	*30%*	*-803*	*77%*	*5280*	*54%*	*-1180*	*79%*
DUT	7074	6%	-3440	1%	10992	4%	-5908	3%
Mean	7524		-3469		11504		-5747	
Standard Deviation	2072		208		2057		283	
Coefficient of Variation	28%		6%		18%		5%	

Notes:
1. Mean, Standard Deviation and Coefficient of Variation exclude LRCF value.
2. d – Deviation from the Mean for a single value.

During the two earlier test phases, where tests were conducted on actual road construction material, it was concluded that the radial measurements were less accurate than the axial measurements. However, during this test phase this is clearly contradicted. It is therefore concluded that the inaccuracies in radial measurement when testing road construction materials is due to the specimens rather than the instrumentation. The possible causes for these inaccuracies are:

- Specimen manufacture differences, for example single layer or multi-layer
- Methods of compaction, vibration or tamping and full-face or smaller;
- The methods of fixing the instrumentation to the specimens.

A further conclusion for this is that the axial displacement measurement is more accurate with higher loads whereas the radial measurement was not affected by load magnitude.

<u>Resilient Modulus and Poisson's Ratio</u>

In order that the variations in the stress could be considered as a function of the variations in the strain the resilient modulus and the Poisson's ratio were calculated as shown in Table 7-21. These material parameters or characteristics were calculated at a particular stress value, based on the standard used at LRSB, of p_r = 250 kPa and q_r/p_r = 2. Again, the LRCF values were low, as shown by the deviation from the mean, and thus have been excluded from the mean, standard deviation and coefficient of variation.

Table 7-21 Resilient Moduli and Poison's Ratio for the Artificial Specimen

Apparatus	**Test 2**				**Test 3**			
	Mr	**d**	**ν**	**d**	**Mr**	**d**	**ν**	**d**
LNEC	71	4%	0.49	0%	64	7%	0.53	4%
UNOT1	89	31%	0.53	8%	72	21%	0.54	8%
UNOT2	44	35%	0.44	10%	45	24%	0.44	14%
LRSB	68	1%	0.50	1%	58	3%	0.49	2%
LRCF	24	65%	0.45	8%	22	63%	0.42	17%
DUT	68	0%	0.50	1%	60	0%	0.52	4%
Mean	68		0.49		60		0.51	
Standard Deviation	16		0.03		10		0.04	
Coefficient of Variation	24%		6%		16%		8%	

Notes:
1. Mean, Standard Deviation and Coefficient of Variation exclude LRCF data.
2. d – Deviation of the particular value from the mean
3. Mr – Resilient Modulus (MPa).
4. ν - Poisson's Ratio

The Poisson's ratio for this material is expected to be 0.5 and the results confirm this. The coefficient of variation for the Poisson's ratio is better than that for the resilient modulus, since Poisson's ratio is more dependent on the radial strain measurements, which were found to have a lower variation than the axial strains. This too is to be expected. The variation for resilient modulus is better for higher stresses, again confirming the conclusions above.

7.6 PHASE 5 - THE PRINCIPAL TEST PROGRAMME

The principal test programme (Test Programme III) was established in order to determine the behaviour of typical soils and unbound granular materials representing those used in foundations of pavements and in the base layers of flexible pavements respectively in Europe. Based on the findings of the three earlier test phases, discussed above, the two test procedures were compiled. Details of the test procedure are contained in Appendix D.3 and are summarised in Table 7-22 and Table 7-23.

The third test programme was conducted at two laboratories (LRSB and LNEC) on two unbound granular materials and two subgrade materials as discussed in the procedure. The objective of this Phase was to collect meaningful data about typical road construction materials found in Europe. The results from these tests, which characterise typical road construction materials, are used in the mechanistic analysis of typical pavement structures in the Chapter 9. All of the results for this test programme (Phase 5) are contained in Appendix F.3.

7.7 COMPARISON OF METHODS SPECIMEN MANUFACTURE

Due to the conclusion that the different specimens are producing different recorded strains depending on the method of manufacture a comparison of these compaction methods is discussed here.

7.7.1 Subgrade Soils

Due to the fine grained nature of the clayey materials that comprise subgrade soils the specimens can be relatively small, less than 100 mm diameter, and therefore these specimens are much more easily handled than the larger granular base specimens. During this work the tamping method of compaction was found to achieve the specified densities at the required moisture contents. However, since particular specimen densities were required for specified moisture contents it was often necessary to vary the compactive effort experimentally until the correct density was achieved. At LNEC an apparatus was used to confirm that the density was consistent throughout the specimen. A nuclear density meter measured the relative density of the specimen as it was spiralled slowly down past the point of measurement {Gomes Correia (1985)}; this apparatus is shown in Photograph 7-1.

Table 7-22 Test Procedure III for the Subgrade Soils

Summary of the Third Test Programme - Subgrade Soils

Aim: To characterise the resilient and permanent behaviour of different subgrade on which European roads may be constructed.

Material: Moisture and Density

Material	Test Moisture Content			Density
	M1	M2	M3	
LOC	Sr = 70%	Sr = 80%	Sr = 90%	0.90 dd
BSC	Wopt - 2%	Wopt - 1%	Wopt	1.00 dd
LIM	Sr = 70%	Sr = 80%	Sr = 90%	0.97 dd
LIR	Sr = 70%	Sr = 80%	Sr = 90%	0.97 dd
SFB	W = 4%	Dry	------	1.00 dd

All tests are to be conducted on two identical specimens

Loading: Haversine wave form, frequency 1 second loading 1 second rest.

Test: A. Conditioning - (80 000 cycles)

	σ_3 min =	10 kPa	σ_3 max =	10 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(60% q_f) kPa.	

B. Resilient Deformation I - (50 cycles/ stress path)

1	σ_3 min =	10 kPa	σ_3 max =	10 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(50% q_f) kPa.	
2	σ_3 min =	10 kPa	σ_3 max =	===========>	(VCP) qr/pr = 1.5
	q min =	0 kPa	q max =	(50% q_f) kPa.	

C. Resilient Deformation II - (50 cycles/ stress path)

1	σ_3 min =	30 kPa	σ_3 max =	30 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(50% q_f) kPa.	
2	σ_3 min =	30 kPa	σ_3 max =	===========>	(VCP) q_r/p_r = 1.5
	q min =	0 kPa	q max =	(50% q_f) kPa.	

D. Resilient Deformation III - (50 cycles/ stress path)

1	σ_3 min =	45 kPa	σ_3 max =	45 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(50% q_f) kPa.	
2	σ_3 min =	45 kPa	σ_3 max =	===========>	(VCP) q_r/p_r = 1.5
	q min =	0 kPa	q max =	(50% q_f) kPa.	

E. Permanent Deformation - (80 000 cycles)

1	σ_3 min =	10 kPa	σ_3 max =	10 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(65% q_f) kPa.	
2	σ_3 min =	10 kPa	σ_3 max =	10 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(50% q_f) kPa.	
3	σ_3 min =	10 kPa	σ_3 max =	10 kPa	(CCP) q_r/p_r = 3.0
	q min =	0 kPa	q max =	(35% q_f) kPa.	
4	σ_3 min =	30 kPa	σ_3 max =	30 kPa	(VCP) q_r/p_r = 1.5
	q min =	0 kPa	q max =	(65% q_f) kPa.	
5	σ_3 min =	30 kPa	σ_3 max =	30 kPa	(VCP) q_r/p_r = 1.5
	q min =	0 kPa	q max =	(50% q_f) kPa.	
6	σ_3 min =	30 kPa	σ_3 max =	30 kPa	(VCP) q_r/p_r = 1.5
	q min =	0 kPa	q max =	(35% q_f) kPa.	

Where: CCP - Constant Confining Pressure
VCP - Variable Confining Pressure

Table 7-23 Test Procedure III for the Unbound Granular Materials

Summary of the Third Test Programme - Unbound Granular Materials

Aim: To investigate the properties of certain materials commonly used in road construction throughout Europe.

Material:

Material	Dry Density (kg/m^3)	Moisture Contents T1 (%)	T2 (%)	P (%)
Soft Limestone (CCT)	2370	2.0	4.3	2.0
Hard Limestone (CCD)	2250	3.5	4.5	3.5
Microgranite (MIG)	2150	3.3	4.3	3.3
Dresden N. Grav. (DN(	2000	3.5	5.0	3.5

Compaction Methods: Vibrocompaction

Test: 1. Resilient Test (conducted on a single specimen)
Four specimens at two different moisture contents (T1 amd T2).

A. Conditioning.

1 σ_3 min = 0 kPa σ_3 max = 100 kPa (VCP) q_r/p_r = 2.5
q min = 0 kPa q max = 600 kPa.

B. Resilient Deformation I - (VCP) S3 min = 0 kPa

1 σ_3 min = 0;15 kPa σ_3 max = 50;100;175;250 kPa(VCP) q_r/p_r = 0.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 30;60;105;150 kPa

2 σ_3 min = 0;15 kPa σ_3 max = 50;100;150;200 kPa(VCP) q_r/p_r = 1.5
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 150;300;450;600 kPa

3 σ_3 min = 0;15 kPa σ_3 max = 30;60;100 kPa (VCP) q_r/p_r = 2.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 180;360;600 kPa

4 σ_3 min = 0;15 kPa σ_3 max = 10;15;20 kPa (VCP) q_r/p_r = 2.0
At each confining pressure the following deviator stresses are applied:
q min = 0 kPa q max = 150;225;300 kPa

2. Permanent Test (each stress path is conducted on a new specimen)
Four specimens at the same moisture content.

1 σ_3 min = 0 kPa σ_3 max = 20 kPa (VCP) q_r/p_r = 2.5
q min = 0 kPa q max = 300 kPa.

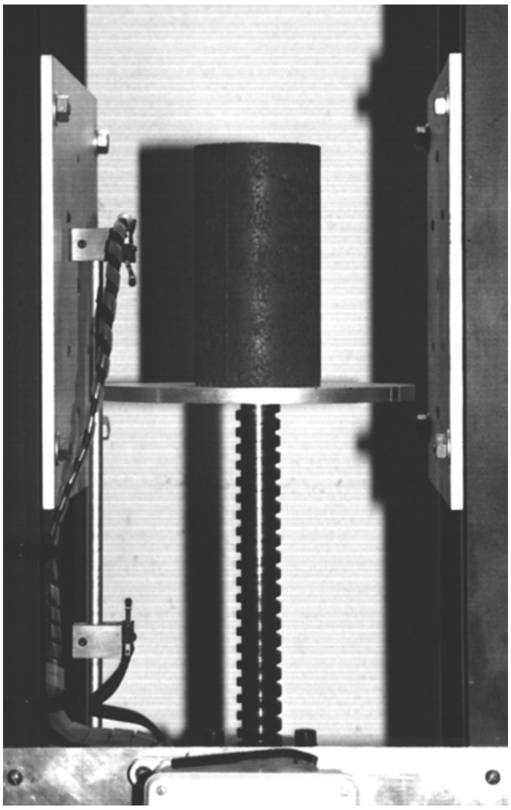
2 σ_3 min = 0 kPa σ_3 max = 100 kPa (VCP) q_r/p_r = 2.5
q min = 0 kPa q max = 600 kPa.

3 σ_3 min = 0 kPa σ_3 max = 200 kPa (VCP) q_r/p_r = 1.5
q min = 0 kPa q max = 600 kPa.

4 σ_3 min = 0 kPa σ_3 max = 75 kPa (VCP) q_r/p_r = 1.7
q min = 0 kPa q max = 300 kPa.

Where: CCP - Constant Confining Pressure
VCP - Variable Confining Pressure

Photograph 7-1 Specimen Density Measurement at LNEC



Once compacted the strain measurement instrumentation can be successfully attached to the specimen by using the cruciform vanes, described earlier, which are easily pressed into the specimen. Due to the soft nature of these specimens, however, a frame is used to support the instrumentation in order that it does not 'hang' on the specimens causing deformation.

7.7.2 Unbound Granular Materials

The compaction of granular materials is more complex due to the less cohesive nature of the materials. Instrumentation cannot be pressed into the specimen and must therefore be either 'cast' into the specimen during compaction or fixed to the specimen after manufacture. During this project specimens were manufactured in multi-layers using both tamping and vibrating hammers. It was found that specimens manufactured by the tamping method were often not dense enough. The vibrating hammer method gives better density due to shear forces applied. At UNOT specimens were compacted using a vibrating table and a surcharge applied to the material. The material was placed in the mould in five layers and compressed. At

LRSB, however, a specifically manufactured apparatus compacted the specimen in a single layer to a particular density, thus eliminating the iterative method of attaining the correct density for particular moisture contents. However, when the material is compacted in a single layer there are some questions about the uniformity of the density of the material throughout the specimen. LRSB checked this using a nuclear device similar to that at LNEC described above. Both UNOT and LRSB use a full-face plate with their vibrating methods, whereas at LNEC a smaller diameter plate was used to compact up to eight layers. This method induces more shear into the material and therefore greater densities are possible.

When the instrumentation studs are fixed to the mould, as is necessary for unbound granular material, in order that they are embedded in the specimen, the studs inhibit the compaction of the material directly around the studs. If the material is compacted in layers the material can be specifically compacted around the studs. When the material was compacted in a single layer substantial voids were found around the studs. At LRSB these voids were filled with a cement mortar that fixed the studs in position.

7.8 SUMMARY

After assessing the results of Phase 1 the test procedure was modified and a series of tests were conducted by each of the four laboratories on similar materials (Phase 2). Again different results in measured strain were obtained and it was decided to exclude all possible influences related with testing of natural materials and to test an artificial specimen (Phase 3). This was followed by the final phase, in which different materials were tested in different laboratories in accordance to a common procedure, in order to determine the characteristics of common road construction materials in Europe (Phase 5). This extensive test programme has yielded sufficient data to enable a detailed analysis of the results using mathematical models that attempt to describe the behaviour of the materials under traffic loading conditions.

In order to obtain repeatable results a well-prescribed test procedure is necessary. The test procedure should be as simple as possible removing all unnecessary actions that might introduce operator peculiarities.

A reliable compaction method, producing homogeneous densities, is needed for the preparation of the specimens. It was found that the magnitude of the permanent strain (axial and radial) that occurred during the conditioning was dependent on the method of compaction of the specimen, thus there is a need for a standardised compaction method for triaxial specimens. The vibrocompression method used at LRSB, for example, is fast (1 minute to compact a specimen), largely automated (which reduces the operator's influence), and produces very homogeneous specimens in density because the compaction is performed in one layer. For the compaction of fine-grained materials often the simple method of tamping multiple layers is used.

In summary it is clear that not only is there a large variation in the strains from specimens tested in a single laboratory but the tests on the artificial specimen show that there is also some discrepancy between laboratories. It is also apparent that there is a substantial variation in the loads (stresses) applied to the specimens that will have an obvious effect on the strains. It is, therefore, necessary to take this difference in stress into consideration when comparing results.

However, large differences in the results are thought to be due to differences in the compaction method used by the laboratories. Methods of compaction which induce high levels of shear, such as the vibrating hammer (LNEC) and manual tamping under cyclic preloading (DUT) result in lower permanent strains than the methods where the compaction is full face and tend not to induce such high levels of shear, such as the vibrocompression apparatus (LRSB) and vibrating table and full face static load (UNOT).

The mean radial measurement for three specimens each manufactured and tested at the four laboratories is -140 $\mu\varepsilon$, the standard deviation is 68 $\mu\varepsilon$ and the Coefficient of Variation is 49%, which is a great deal poorer than the 12% for the axial resilient strain measured between the laboratories. It is thus surmised that there are greater errors in the radial measurement systems than the axial measuring systems, substantiating that found during the earlier test programme.

Larger specimens give less variable results, however large specimens require more material, are more time consuming to fabricate, more difficult to manoeuvre and the apparatus required to test them is much larger thus more expensive. Another disadvantage of very large specimens is that they generally are not suitable for performing variable confining pressure tests since they use internal vacuums.

For the unbound granular material the axial resilient strain results show little systematic difference although improved readings appear to result from larger specimen size. The variability in readings is particularly high for the UNOT tests (which may be due to stud rotation generating apparent strain - sometimes increasing, sometimes decreasing the measured values above the average obtained at all the laboratories). For radial resilient strains all laboratories yielded a large scatter in strain values. There was no systematic variation in radial strains between laboratories.

Experiments with various on-sample instruments for measuring the axial and radial strain of soil and aggregate specimens subjected to repeated load triaxial testing (at different sizes at different laboratories) have been described. Results frequently differ, but the origin of these differences is often unclear. After the completion of the test programme on the artificial test programme some calibration of the various apparatus and instrumentation was conducted and each laboratory's apparatus was harmonised for the main test programme (Phase 5).

A number of methods of analysis will be tested and recommendations made as to how this type of data should be analysed and what the expected errors due to test apparatus and instrumentation and sample manufacture are. The significance of these errors will be tested in a mechanistic pavement design method and recommendations made.

8 ANALYSIS OF THE BEHAVIOUR OF THE MATERIALS BY ANALYTICAL MODELLING

8.1 INTRODUCTION

In chapter 5 the models that are to be used to describe the behaviour of the road construction materials were discussed. During this chapter the results of the tests from the three test phases, Phase 1, 2 and 5, are analysed and the results discussed.

Briefly, as stated in Chapter 5, the models chosen are as follows:

Models for all Road Construction Materials

a. Simple Linear Elastic Model

b. The k-theta model

c. The Uzan Model

For Fine Grained Subgrade Soils used in Road Construction

d. The Brown Model

e. The Loach model

For Unbound Granular Materials used in Road Construction

f. The Boyce model

g. The Mayhew model

8.2 DATA VERIFICATION AND MANIPULATION

After conducting the test and 'capturing' the data (stresses and strains) by some method, it is necessary to appraise the data and to undertake data verification. The data verification should exclude any erroneous results. Care, however, must be taken that true results are not excluded, even if they appear erroneous. For example, as has been shown, erroneous data occur as a result of small stress applications and thus small strain measurements and these should be excluded from any modelling, since they will effect the overall characterisation of the material.

A problem with data verification is that is it somewhat subjective. It is with this in mind that a method was devised for this work, as described below.

8.2.1 Initial Screening (Removal of Obviously Poor Data)

The test results, as presented in the previous chapter, comprise spreadsheets containing the stresses and strains for each stress path applied for each specimen tested. The resilient modulus and the Poisson's ratio are calculated for each stress path. An initial screening pass is made that eliminates all data from any particular stress path that does not comply with the following criteria:

- Poisson' ratio $0 < \nu < 1$
- Experimental resilient modulus $M_r > 0$
- Repeated deviator stress $q_r > 0$
- Final mean normal stress $p_2 > 0$

Strictly speaking all materials should have a Poisson's ratio of between 0 and 0.5. This is because any material with a Poisson's ratio of less than 0 would be collapsing within itself, which would be clearly impossible for a compacted material. Some dilation is, however, possible due to particles 'rolling' over one another therefore the maximum Poisson's ratio is taken to be 1. It is impossible for a material to have a negative resilient modulus. The deviator stress can be negative when *in-situ*, but this should not occur for conventional compressive triaxial testing. Similarly, the maximum mean normal stress will always be positive for triaxial conditions.

The coefficients of the constitutive relationships (models) are calculated from the results of the test data using the analytical methods described in this chapter. Once the model coefficients have been established it is possible to calculate a predicted resilient modulus value for each stress path, and for each relationship, and plot these against the experimental values calculated for the particular stress path. An example of this is shown graphically in Figure 8-1 for an unbound granular material and in Figure 8-2 for a subgrade soil.

Figure 8-1 Comparison of the Experimental and Modelled Resilient Modulus for an Unbound Granular Material

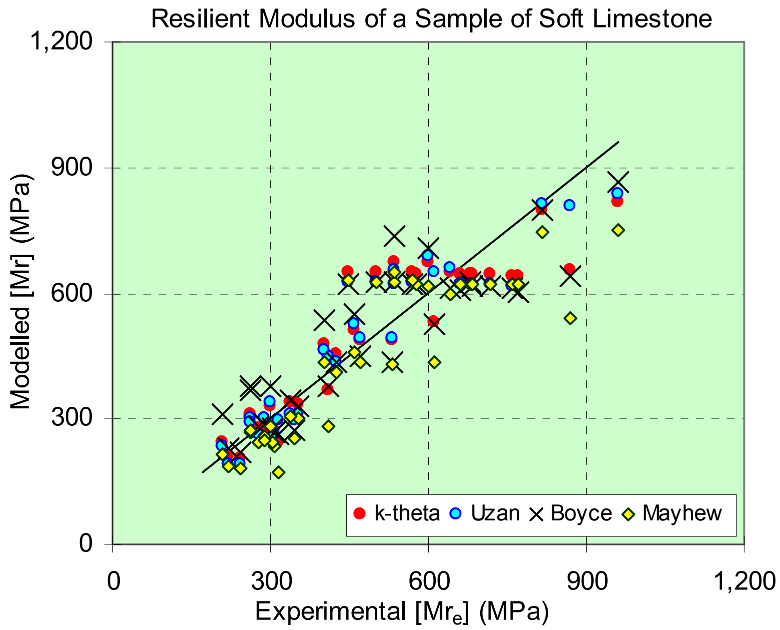
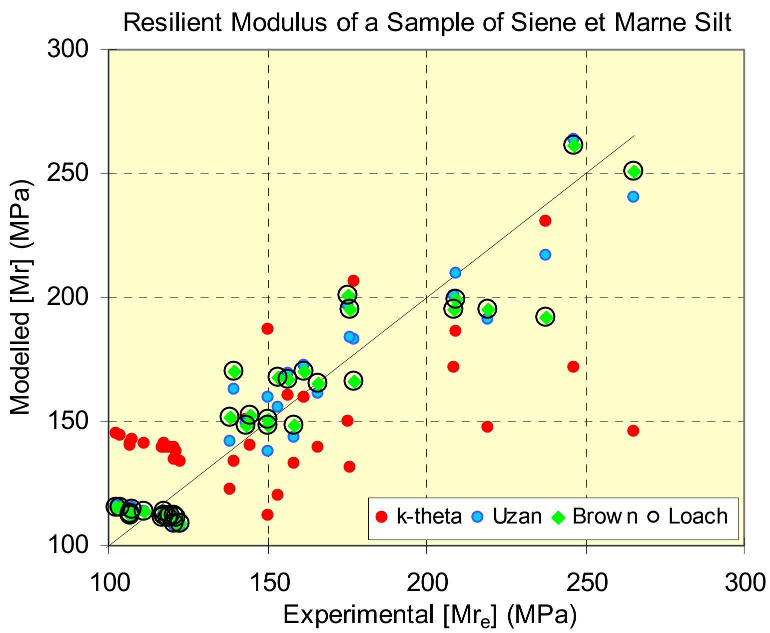


Figure 8-2 Comparison of the Experimental and Modelled Resilient Modulus for a Subgrade Soil



8.2.2 Secondary Screening (Removal of Outliers by Percentile)

The data is then screened to exclude the data from all stress paths that are outside certain variations from the average. These values are termed as outliers. For this study outliers are defined as those values outside the 95th percentile (95%'ile) of the average (across all relevant models) of the absolute difference between the experimental and modelled values for resilient modulus, shown by the equation:

$$Outlier \geq 95\%\ ile\left[Average\left|Mr_e - Mr\right|\right] \qquad \text{Eqn.8-1}$$

Where $\left|Mr_e - Mr\right|$ is the absolute difference of the experimental and the modelled Resilient Modulus.

There are instances where a particular stress path is considered to be an outlier when analysed by one particular model and not by the others. It is for this reason that the average is used and the 'erroneous' data is excluded for all models. Therefore the data is always exactly the same when applied to the different models for each analytical sequence in the removal of outliers.

Once the outliers have been removed from the data, the parameters for each model are calculated again and the modelled resilient modulus recalculated from this data. This procedure of identifying outliers can be reapplied to the new data and so on. If a vast sample size (which is normally distributed) were used then it would be expected that one could continue to remove 5% of the worst results almost indefinitely, however these data sizes are limited and the removal of data makes a significant difference to the outcome of the analysis.

It is necessary to identify the data and results for each level from which the outliers have been removed. For this work the designation used for the original data (after the initial screening) is termed *100%'ile Data.* After the first level of outliers has been removed, corresponding to the 95th percentile of the data, the remaining sample is called the *95%'ile Data*. The next level is in fact the 95%'ile of the previous 95%'ile but for clarity is termed the *90%'ile Data* and so on. This is laid out in Table 8-1 and as can be seen was conducted 4 times resulting in *80%'ile Data*.

This was not conducted for all of the results for all of the test programmes. Due to the large scatter of the test results obtained during Test Programme I this method of removing the outliers was applied and the results applied to the other two test programmes.

Table 8-1 Removal of Poor Data and Outliers from the Test Data

Start Data	Data Removed	Designation
All Test Data	Remove outliers from the data as per initial screening criteria (Table 8-4)	Termed: *100%'ile Data*
Removal of 1st level Outliers	Outliers values falling outside the **95th percentile** of **100%** are removed	Termed: *95%'ile Data*
Removal of 2nd level Outliers	Outliers values falling outside the **95th percentile** of **95%** are removed	Termed: *90%'ile Data*
Removal of 3rd level Outliers	Outliers values falling outside the **95th percentile** of **90%** are removed	Termed: *85%'ile Data*
Removal of 4th level Outliers	Outliers values falling outside the **95th percentile** of **85%** are removed	Termed: *80%'ile Data*

8.2.3 Analytical Modelling Methods Used to Model the Results

Two different analytical methods were used to analysis the data. The analysis was started with the simpler subgrade soil models using a proprietary software package. When this method was applied, however, to the more complex unbound granular material models, it was found to not always provide realistic results (non-convergence) and therefore a second method of analysis was employed for the granular materials.

- The first method used for the analysis of the results obtained from subgrade soils was a non-linear regression analysis computer program called NLREG {Sherrod (1998)}. This program performs a statistical regression analysis that estimates the values of the model coefficients for general non-linear functions and allows the function to best fit the observed data.

- The second method, used to analyse the results of the testing of unbound granular materials, was that of the method of least squares. The equations were entered into a *Microsoft Excel* spreadsheet and the *Solver* function used to determine the minimum value of one cell (sum of the squares) by changing the values in a number of other cells (the model coefficients).

The advantage of the first method is that the software calculates some statistical values, which give some indication of the accuracy of the results and the relevance of particular parameters (these were discussed earlier). This information can also be calculated using formulae in a spreadsheet, of course, but this leads to very complicated spreadsheets. In reality these accuracy indications are only of real benefit to those interested in creating and modifying the models. Since this work is not involved in the improvement of models but the application of existing models to test results and pavement design it is considered that a simple regression coefficient of correlation (or correlation coefficient) should suffice.

8.3 PRESENTATION OF THE RESULTS

Each of the specimens tested during the three Test Programmes has been analysed, using the relevant models and the results are presented for each specimen in Appendix G containing a total of 128 different curve fitting analyses. Examples of the presentation of the analysis for subgrade soils and the unbound granular materials are shown in Table 8-2 and Table 8-3. The characteristic resilient modulus is calculated for the subgrade soils and for two of the four models used to analyse the granular materials, whereas four material parameters (M_r, ν, ε_s and ε_v} are calculated for the remaining two models used to analyse the granular materials.

8.3.1 Modelling Analyses to determine the Material Coefficients

With reference to both Table 8-2 and Table 8-3 the top line describes the test programme, the laboratory where the test took place, the material and its code, and the percentile value by which data was reduced, (outliers removed). Beneath this are the following sub-headings:

<u>Experimental Data</u>

Shown here is the sample size (number of stress paths) and any model constants (coefficients) as described previously. For subgrade soils the suction value is shown at the specific moisture content, at which the test was conducted. For unbound granular materials, the material parameter p* defined at the specific moisture content, at which the test was conducted is shown. The experimental values of resilient modulus (Mr_e) and Poisson's ratio (ν_e) are calculated at the characteristic deviator stress shown. It should be noted that the characteristic values of resilient modulus, Poisson's ratio, volumetric and shear strain, calculated in the 'Characteristic Values' block lower in the table, are calculated using this characteristic deviator stress and an associated mean normal stress.

Modelled Data

The model coefficients are calculated for each relationship as well as the correlation coefficient for the particular fit. It is possible to determine in a qualitative manner how well a model describes the relationship between variables or experimental data. There is a ratio of the explained variance to the total variation that is called the coefficient of correlation (or correlation coefficient). Since this ratio is always positive it is denoted by r^2 and varies between 0 (very poor correlation) and 1 (very good correlation).

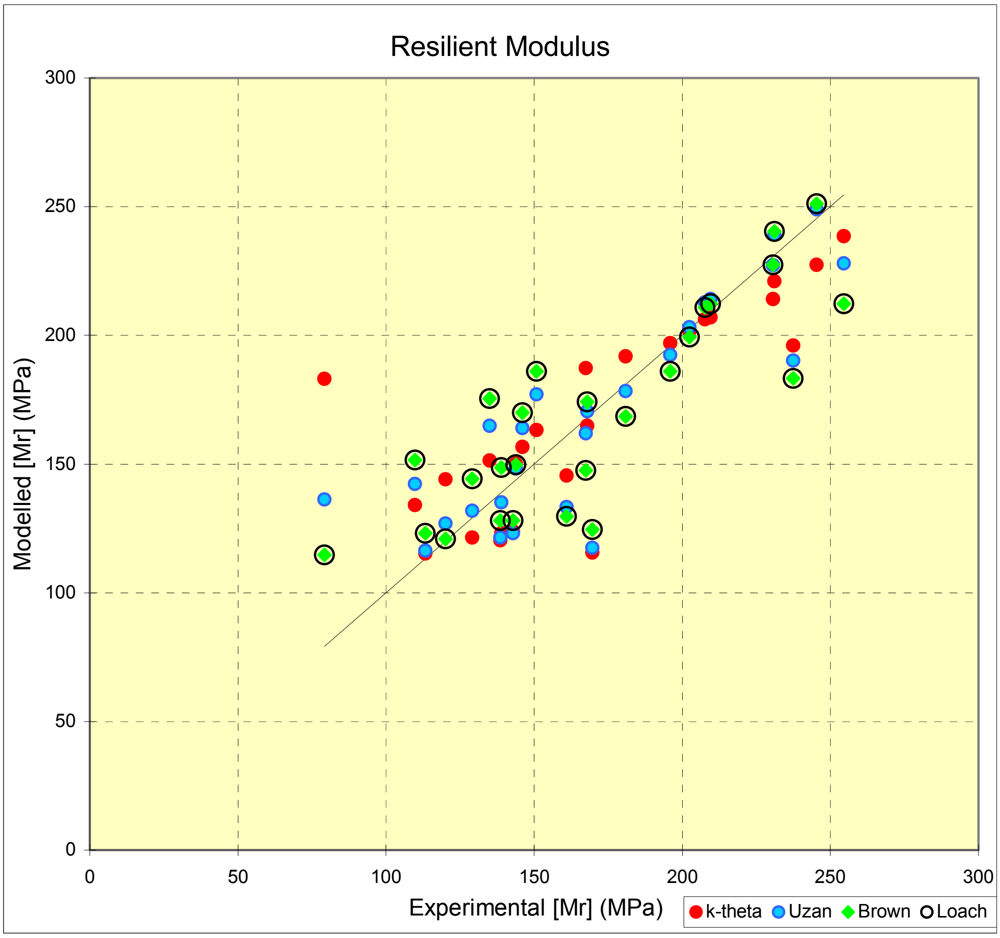
Using these model coefficients, the resilient modulus can be calculated for each stress path and plotted against the experimental resilient modulus and this is shown graphically for both material types. When the data is analysed using the Boyce and the Mayhew models, for the unbound granular materials, it is also possible to calculate the Poisson's ratio and the volumetric and shear strains for each of the stress paths and these too are compared graphically against the experimental data.

It was shown earlier that both the Boyce and the Mayhew models attempt to model the volumetric and shear strains separately. The Boyce model, however, shares the G_a material coefficient between these two components and, when modelled, two different values are obtained for the same material coefficient. In order to prevent this, the two equations for strain have been substituted in the equation for resilient modulus and the best-fit analysis is conducted in this manner. This is not necessary for the Mayhew model since there are no common model coefficients in the two strain parameters and this relationship is analysed as two separate strain components. Thus there are two correlation coefficients, one for each of the two strain components.

Table 8-2 Example of the Presentation of the Model Analysis for Subgrade Soils

TP-I : UNOT ***Fountainebleau Sand (SFB)*** ***100% Percentile Data***

Experimental Data	Sample Size: 15 Suction (s) = 3 kPa @ ω= 0%			
	Chara'tic Values at q_2 = 26 kPa		Mr_e= 152 MPa	υ_e = 0.58
Modelled Data	**k-theta**	**Uzan**	**Brown**	**Loach**
	k_1 = 144,790	k_3 = 227,609	A = 92,013	C = 3,067,105
	k_2 = 0.3741	k_4 = 0.1647	B = -0.2548	D = 0.7452
		k_5 = 0.1651		
	r^2= 0.635	r^2= 0.750	r^2= 0.710	r^2= 0.710
Characteristic Values		At:	p_2 = 12 kPa	& q_2= 26 kPa
	k-theta	**Uzan**	**Brown**	**Loach**
	Mr_c= 97 MPa	Mr_c= 127 MPa	Mr_c= 159 MPa	Mr_c= 159 MPa



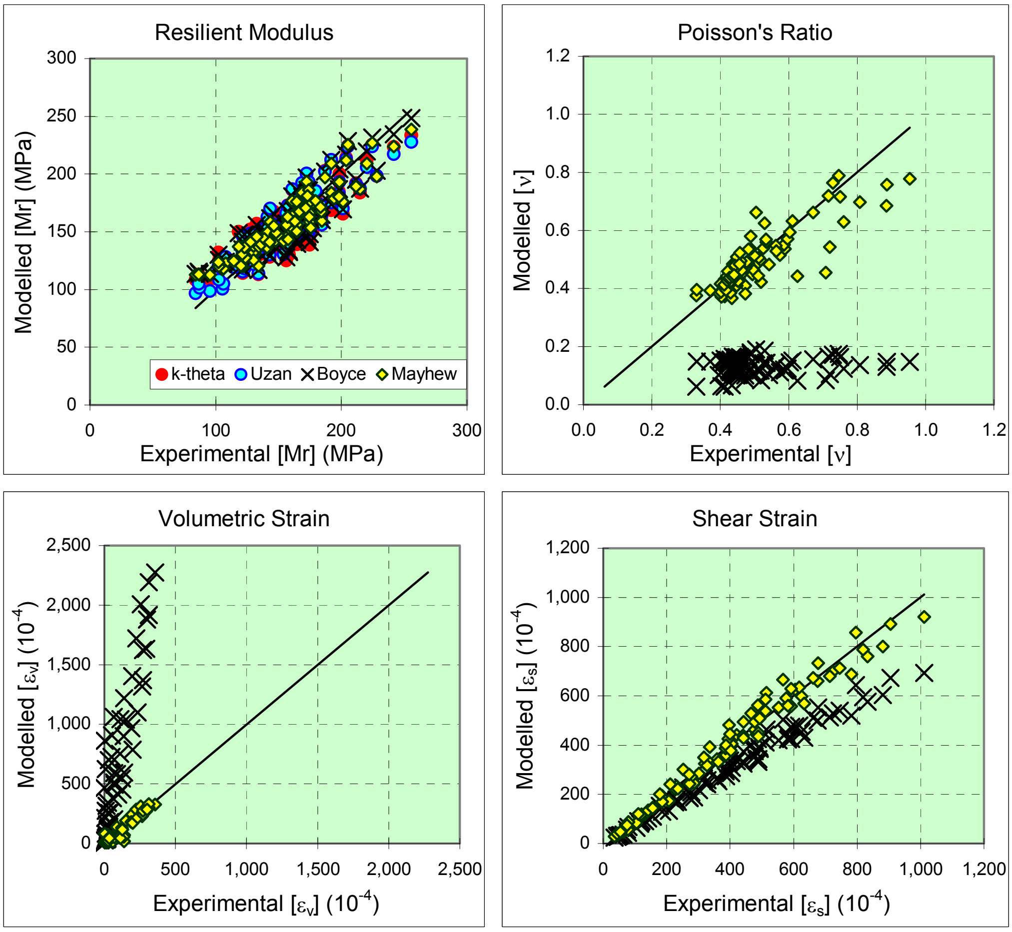
Exactness of the estimated parameter value									
Model	**k-theta**		**Uzan**			**Brown**		**Loach**	
Parameter	k1	k2	k3	k4	k5	A	B	C	D
Std.Error=	6990	0.06	8579	0.09	0.06	8750	0.03	291652	0.03
t=	20.71	6.39	26.53	1.87	2.85	10.52	-7.58	10.52	22.17
Prob(t)=	0%	0%	0%	7%	1%	0%	0%	0%	0%

The smaller the standard error, the more confident one can be that the parameter's value matches its estimated value.
The larger the absolute value of t, the less likely that the actual value of the parameter could be zero.
The smaller the value of Prob(t), the more significant the parameter and the less likely that the parameter value is zero.

Table 8-3 Example of the Presentation of the Model Analysis for Unbound Granular Materials

TP-I : LRCF(2) Fountainebleau Sand (SFB) 80% Percentile Data

Experimental Data	Sample Size: 90	p^*= 15 kPa @ ω= 0.0%		p_a'= 100 kPa
	Characteristic Values at q_2 = 54 kPa		Mr_e= 150 MPa	υ_e = 0.51
Modelled Data	**k-theta**	**Uzan**	**Boyce**	**Mayhew**
	k_1 = 122,540	k_3 = 178,132	Ga = 64,387 kPa	Ga = 48,811 kPa
	k_2 = 0.3034	k_4 = 0.1504	Ka = 46,148 kPa	Ka = 124,479 kPa
		k_5 = 0.1218	n = 0.5331	n = 0.5406
				β = 0.6475
				m = 0.5443
	r^2= 0.630	r^2= 0.689	r^2= 0.660	$r^2\varepsilon_v$= 0.734 $r^2\varepsilon_s$= 0.888
Characteristic Values	At p_1= 6 kPa		p_2 = 15 kPa and	q_1= 4 kPa q_2= 54 kPa
	k-theta	**Uzan**	**Boyce**	**Mayhew**
			ε_{vc}= 237 ε_{sc}= 341	ε_{vc}= -485 ε_{sc}= 444
	Mr_c= 97 MPa	Mr_c= 125 MPa	Mr_c= 103 MPa	Mr_c= 319 MPa
			υ_c= 0.06	υ_c= 3.26



Characteristic Values

The characteristic stresses, as defined earlier, are shown in this section of the table and the characteristic resilient modulus is calculated for each model. The characteristic volumetric and shear strains and the characteristic Poisson's ratio are calculated for the Boyce and Mayhew unbound granular material models. Because ε_s and ε_v are calculated for the Boyce and Mayhew models is possible to calculate the characteristic Poisson's ratio for these two models and this is done.

It is noted, in the analysis of the granular material that the Boyce model does not characterise the Poisson's ratio or the volumetric strain well and this is a failing of the model. Since the Poisson's ratio is closely related to change in volume, thus volumetric strain, it is expected that if a poor correlation were found for one it would also occur for the other. Having said this it is noted that the Mayhew model estimates the characteristic Poisson's ratio at well above 3 and this is clearly erroneous.

Quality of the Estimated Model coefficients for Subgrade Soils

For the subgrade soils the standard error, t and Prob(t) statistical values are shown for each material coefficient. These are an output of the software package used to analyse these results as discussed. A detailed explanation for each of these statistical values is made in Chapter 6. Because the model analysis for the unbound granular materials is conducted manually, a more simple correlation for the each model is made.

8.3.2 Analysis of the Test Results and Comparison Method

Having conducted the testing and measured the strains, under predefined stress conditions, some discussion must take place with regard to how these values will be used in pavement design. After all, the purpose of the material testing and the subsequent analysis of the results is to establish useful and verified parameters, of which the accuracy is quantified, and which can be used to design more economic pavement structures.

From the three Test Programmes conducted it is possible to make the following comparisons:

- Test Programme I
  - a) To compare the original test results against those results that have been cleansed (by reducing the data by outliers - data exceeding specific variation limits;
  - b) To compare the results for the same material tested at different laboratories;
  - c) To compare the results of the same data using two different analytical methods. A dry sand was tested which can be considered to be classified as a subgrade soil as well as an unbound granular material;
- Test programme II
  - d) To compare the results for the same material tested at different laboratories;
  - e) To compare the results for the same material tested at a single laboratory;
- Test Programme III
  - f) To compare the results for the same material tested at a single laboratory.

Further, the model coefficients for each specimen, and thus the material, are determined, which provides a range of material parameters that can be used in pavement design.

During the design of a pavement structure, using complex relationships and analytical methods to describe the material behaviour, it is the independent variables, which are entered into the computer program, that are important. These variables are the characteristic resilient modulus and Poisson's ratio (material parameters at characteristic stresses) for simple models and the model coefficients in the case of the more complex models. It is quite difficult to appreciate the magnitudes of the coefficients, due to the variation of the numbers (up to 10^7 times) and as such it is much more convenient to consider the accuracy of the material parameters (characteristic resilient modulus and Poisson's ratio).

During the analysis it was observed that from time to time the results obtained from the model coefficients were unrealistically large. Without investigating the intricacies of the relationships in great detail it is surmised that this is because these relationships are multi-dimensional and they simulate multi-dimensional valleys and hills. During the search for the minimum value, or solution, the model gets 'stuck' in an incorrect valley and thus an incorrect solution is found (lowest point in that valley) without ever reaching the correct valley. When this happened the analysis was often retried with different initial conditions but most times no reasonable solution was found since the correct 'valley' was never located. Under these circumstances, it was decided that the results should be omitted from the overall analysis and, in order to quantify this, a set of basic rules for the model coefficients was formulated as shown in Table 8-4.

It must be noted that the coefficient minimum and maximum values have been chosen from experience gained during the analysis of the data and thus these values are somewhat pragmatic. It was noted that as certain values were exceeded the models tended to produce outrageous parameters and thus limiting values were selected. The maximum and minimum values for resilient modulus and Poisson's ratio were chosen as reasonable limits, based on the discussions in Chapter 2.

Table 8-4 Limiting Criteria for the Parameters and Model Coefficients

Parameter and Material Coefficient		Subgrade Soils		Unbound Granular Materials	
		Minimum	Maximum	Minimum	Maximum
k-theta	k_1	0	1,000,000	0	2,000,000
	k_2	-1	1	-1	1
Uzan	k_3	0	1,000,000	0	1,000,000
	k_4	-1	1	-1	1
	k_5	-1	1	-1	1
Brown	A	0	1,000,000	NA	NA
	B	-1	1	NA	NA
Loach	C	0	4,000,000	NA	NA
	D	0	2	NA	NA
Boyce	Ga	NA	NA	0	1,000,000
	Ka	NA	NA	0	1,000,000
	n	NA	NA	0	2
Mayhew	Ga	NA	NA	0	1,000,000
	Ka	NA	NA	0	1,000,000
	n	NA	NA	-1	2
	b	NA	NA	0	1
	m	NA	NA	0	2
Resilient Modulus	M_r	0	500	0	2,000
Poisson's ratio	ν	0	1	0	1

NA – Not applicable

8.3.3 Actual Removal of the Outliers from the Test Results

The tables shown in Table 8-5 and Table 8-6 are examples of the summaries of all of the results summarise all of the results obtained for a particular test programme for a particular material (Test Programme I; Fontainebleau Sand (SFB) Subgrade Soils and Unbound Granular Material, in this case). Each sub-table contains the results of the analysis for the results conducted at all of the laboratories for a particular set of data as the outliers are removed (100% to 80%). The Experimental Values of the resilient modulus and Poisson's ratio for each laboratory, as described above, are presented. Also the characteristic parameters (resilient modulus) and model coefficients for each model are presented and in the case of the unbound granular materials the characteristic Poisson's ratios for the Boyce and Mayhew models are also shown. Complete tables containing summaries for all of the materials tested in all three Phases can be found in Appendix G.

Table 8-5 The Results of Fontainebleau Sand tested in Test Programme I and Analysed as a Subgrade Soil

Test Programme 1 Fontainebleu Sand **Outlier % = 100%**

Laboratory		Experimental Values		Modelled Values												
				k-theta			Uzan				Brown			Loach		
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	A	B	Mr_c	C	D
UNOT	SFB	152	0.58	97	144,790	0.3741	127	227,609	0.1647	0.1651	159	92,013	-0.2548	159	3,067,105	0.7452
LRSB(1)	SFB	164	0.24	86	137,015	0.4351	103	254,003	0.3147	0.1621	182	108,597	-0.2404	182	3,619,889	0.7596
LRSB(2)	SFB	142	0.33	80	125,519	0.4218	97	250,215	0.2898	0.2398	164	86,958	-0.2959	164	2,898,583	0.7041
LRCF(1)	SFB	207	0.40	130	186,822	0.3376	152	302,574	0.2056	0.1621	215	124,188	-0.2542	215	4,139,588	0.7458
LRCF(2)	SFB	135	0.51	84	122,753	0.3526	113	190,386	0.1380	0.2398	139	81,915	-0.2453	139	2,730,501	0.7547
Values	Min	135	0.24	80	123,974	0.4061	97	224,469	0.2618	0.2173	139	79,208	-0.2598	139	2,640,272	0.7402
	10%	138	0.28	82	126,079	0.4037	99	226,959	0.2570	0.2145	147	84,026	-0.2594	147	2,800,877	0.7406
	Avg	160	0.41	96	143,380	0.3842	119	244,958	0.2226	0.1938	172	98,734	-0.2581	172	3,291,133	0.7419
	90%	190	0.55	117	170,183	0.3541	142	267,203	0.1799	0.1682	202	116,376	-0.2566	202	3,879,194	0.7434
	Max	207	0.58	130	186,752	0.3355	152	276,435	0.1622	0.1576	215	124,067	-0.2559	215	4,135,583	0.7441

Outlier % = 95%

Laboratory		Experimental Values		Modelled Values												
				k-theta			Uzan				Brown			Loach		
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	A	B	Mr_c	C	D
UNOT	SFB	153	0.57	100	147,453	0.3691	109	223,342	0.2943	0.0583	159	94,894	-0.2406	159	3,163,118	0.7594
LRSB(1)	SFB	177	0.21	94	145,952	0.4177	97	237,135	0.3903	0.0408	183	111,492	-0.2303	183	3,716,387	0.7697
LRSB(2)	SFB	145	0.32	79	126,858	0.4427	95	246,393	0.3050	0.2179	164	86,696	-0.2952	164	2,889,857	0.7048
LRCF(1)	SFB	201	0.39	125	181,176	0.3481	151	305,821	0.1817	0.0408	205	108,211	-0.2970	205	3,607,044	0.7030
LRCF(2)	SFB	133	0.51	85	122,251	0.3408	114	187,456	0.1236	0.2179	137	81,509	-0.2417	137	2,716,962	0.7583
Values	Min	133	0.21	79	122,689	0.4068	95	218,416	0.3142	0.1429	137	82,022	-0.2425	137	2,734,050	0.7575
	10%	138	0.26	82	125,676	0.4037	96	219,193	0.3122	0.1419	146	85,982	-0.2475	146	2,866,063	0.7525
	Avg	162	0.40	97	144,738	0.3837	113	240,029	0.2590	0.1151	170	96,560	-0.2610	170	3,218,674	0.7390
	90%	191	0.55	115	168,167	0.3591	136	267,368	0.1892	0.0800	196	108,469	-0.2761	196	3,615,648	0.7239
	Max	201	0.57	125	181,187	0.3455	151	284,826	0.1446	0.0576	205	112,407	-0.2811	205	3,746,909	0.7189

Outlier % = 90%

Laboratory		Experimental Values		Modelled Values												
				k-theta			Uzan				Brown			Loach		
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	A	B	Mr_c	C	D
UNOT	SFB	152	0.56	97	145,264	0.3797	112	221,998	0.2618	0.0845	157	94,661	-0.2366	157	3,155,350	0.7634
LRSB(1)	SFB	176	0.21	93	145,532	0.4212	99	241,683	0.3685	0.0706	182	108,437	-0.2400	182	3,614,558	0.7600
LRSB(2)	SFB	152	0.29	84	133,226	0.4306	92	235,553	0.3353	0.1577	159	81,208	-0.3121	159	2,706,926	0.6879
LRCF(1)	SFB	204	0.39	130	185,725	0.3322	160	311,928	0.1545	0.0706	208	108,595	-0.3024	208	3,619,845	0.6976
LRCF(2)	SFB	132	0.51	86	121,213	0.3184	112	180,914	0.1298	0.1577	136	84,584	-0.2208	136	2,819,454	0.7792
Values	Min	132	0.21	84	128,739	0.3937	92	209,607	0.3134	0.1270	136	82,678	-0.2360	136	2,755,944	0.7640
	10%	140	0.24	85	129,798	0.3926	95	213,050	0.3058	0.1248	145	86,070	-0.2430	145	2,869,011	0.7570
	Avg	163	0.39	98	146,192	0.3764	115	238,415	0.2500	0.1082	168	95,497	-0.2624	168	3,183,227	0.7376
	90%	193	0.54	117	169,786	0.3531	141	270,916	0.1784	0.0870	198	106,997	-0.2860	198	3,566,556	0.7140
	Max	204	0.56	130	186,536	0.3366	160	294,884	0.1257	0.0714	208	111,168	-0.2946	208	3,705,588	0.7054

Outlier % = 85%

Laboratory		Experimental Values		Modelled Values												
				k-theta			Uzan				Brown			Loach		
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	A	B	Mr_c	C	D
UNOT	SFB	152	0.56	97	145,371	0.3798	120	222,787	0.2076	0.1235	157	92,475	-0.2465	157	3,082,516	0.7535
LRSB(1)	SFB	173	0.22	93	144,805	0.4181	100	241,431	0.3559	0.0826	180	106,385	-0.2452	180	3,546,176	0.7548
LRSB(2)	SFB	147	0.29	83	130,164	0.4252	91	228,717	0.3310	0.1546	155	80,272	-0.3051	155	2,675,745	0.6949
LRCF(1)	SFB	203	0.40	136	189,596	0.3149	163	310,841	0.1406	0.0826	205	105,816	-0.3084	205	3,527,213	0.6916
LRCF(2)	SFB	131	0.51	85	120,763	0.3269	111	182,644	0.1360	0.1546	136	84,900	-0.2207	136	2,829,993	0.7793
Values	Min	131	0.22	83	126,971	0.3958	91	205,324	0.3052	0.1379	136	82,410	-0.2389	136	2,746,992	0.7611
	10%	138	0.25	84	128,168	0.3944	94	209,853	0.2951	0.1353	144	85,192	-0.2452	144	2,839,717	0.7548
	Avg	161	0.40	99	146,140	0.3730	117	237,284	0.2342	0.1196	167	93,970	-0.2652	167	3,132,328	0.7348
	90%	191	0.54	120	172,009	0.3421	146	272,269	0.1565	0.0995	195	104,856	-0.2899	195	3,495,198	0.7101
	Max	203	0.56	136	190,589	0.3200	163	292,886	0.1108	0.0877	205	108,694	-0.2986	205	3,623,141	0.7014

Outlier % = 80%

Laboratory		Experimental Values		Modelled Values												
				k-theta			Uzan				Brown			Loach		
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	A	B	Mr_c	C	D
UNOT	SFB	154	0.57	97	145,371	0.3798	120	222,787	0.2076	0.1235	157	92,475	-0.2465	157	3,082,516	0.7535
LRSB(1)	SFB	169	0.22	91	143,087	0.4257	100	243,599	0.3437	0.1083	176	98,594	-0.2690	176	3,286,458	0.7310
LRSB(2)	SFB	151	0.28	89	134,197	0.3825	91	215,530	0.3428	0.0912	150	76,987	-0.3095	150	2,566,237	0.6905
LRCF(1)	SFB	207	0.40	145	197,344	0.2920	166	312,307	0.1308	0.1083	206	105,845	-0.3099	206	3,528,164	0.6901
LRCF(2)	SFB	130	0.50	87	121,113	0.3108	114	179,546	0.1180	0.0912	136	86,728	-0.2081	136	2,890,928	0.7919
Values	Min	130	0.22	87	130,915	0.3788	91	200,059	0.3045	0.0998	136	82,060	-0.2387	136	2,735,337	0.7613
	10%	138	0.24	88	131,999	0.3775	94	204,751	0.2942	0.1005	141	84,004	-0.2445	141	2,800,125	0.7555
	Avg	162	0.39	102	148,222	0.3582	118	234,754	0.2286	0.1045	165	92,126	-0.2686	165	3,070,860	0.7314
	90%	192	0.54	126	176,079	0.3250	148	272,307	0.1464	0.1096	194	102,148	-0.2984	194	3,404,928	0.7016
	Max	207	0.57	145	198,356	0.2985	166	295,544	0.0956	0.1127	206	106,321	-0.3108	206	3,544,035	0.6892

Note: Complete tables in larger fonts are contained in the Appendices

For Test Programme I the parameters and model coefficients for each model were checked against these criteria starting at 100th percentile data. If there is a value that did not conform to the limiting values in Table 8-4 then these stress paths were removed. For these data the outliers were removed in levels as described earlier until the 80th percentile data was attained. In the example shown in Table 8-5 in the Outlier=100% (*100%'ile*) data there is one value which does not conform, the C coefficient in the Loach model (shown in red), to the limiting values. This indicates that the next level (*95%'ile Data*) should be tested by removing the relevant outliers. This is done and it can be seen that the values for all of the models comply with the limiting values so no further removal of outliers is necessary. For this analysis (Test Programme I) the action of removal of the outliers is conducted until the 80th percentile data set is achieved. This motivated a study to determine if there was an optimum percentile value for which 'clean' data could be guaranteed.

In the example of the analysis of the unbound granular material shown in Table 8-6 exactly the same philosophy was taken in using the limiting values to determine the removal of outliers to achieved 'clean' data.

It can be clearly seen that more removal of outliers was necessary for this data, if fact even for the 80th percentile data there is a single value of Poisson's ratio that does not comply with the limiting values of Table 8-4. These two analyses are both made on Fontainebleau Sand, which is thought to act as both a granular material and a subgrade soil. Thus the base data (*100%'ile Data*) are identical. Clearly it is more difficult to analyse these data using the complex models for unbound granular than the more simple models for subgrade soils.

Table 8-6 The Results of Fontainebleau Sand tested in Test Programme I and Analysed as a Granular Material

Test Programme I Fountainebleau Sand (SFB) **Outlier % = 100%**

Laboratory		*Experimental Values*		*Modelled Values*																		
				k-theta			*Uzan*				*Boyce*					*Mayhew*						
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	υ_c	Ga	Ka	n	Mr_c	υ_c	Ga	Ka	n	β	m
UNOT	SFB	197	0.30	100	143,888	0.4168	108	232,063	0.3738	0.0434	117	**-0.53**	153,098	29,672	0.5530	144	0.61	86,707	57,039	0.3254	0.1771	0.6414
LRSB(1)	SFB	203	0.22	82	126,933	0.4909	124	256,430	0.3107	0.1795	69	**-0.95**	1,000,000	13,802	0.4358	107	0.22	86,127	63,568	0.3178	0.0460	0.3910
LRSB(2)	SFB	193	0.29	83	122,142	0.4368	121	250,300	0.2896	0.2404	68	**-0.99**	**4,002,065**	13,203	0.4755	113	0.25	76,463	89,646	0.3946	0.0378	0.4461
LRCF(1)	SFB	243	0.44	139	186,822	0.3376	180	302,574	0.2056	0.1794	145	0.10	98,854	72,623	0.4560	119	0.61	89,968	259,650	0.2149	0.4903	1.0281
LRCF(2)	SFB	154	0.51	90	122,753	0.3526	131	190,386	0.1380	0.1646	62	**-0.99**	**3,058,585**	11,908	0.4902	115	0.53	52,484	615,913	0.4948	0.6551	1.4807
Values	Min	154	0.22	82	121,629	0.4378	108	222,031	0.3118	0.1394	62	-0.99	2,792,477	8,335	0.4745	107	0.22	74,738	262,429	0.3668	0.2757	0.7881
	10%	170	0.25	83	121,976	0.4373	113	227,297	0.3013	0.1441	64	-0.99	2,697,725	10,004	0.4751	109	0.24	75,429	253,771	0.3635	0.2767	0.7899
	Avg	198	0.35	99	140,508	0.4069	133	246,351	0.2635	0.1614	92	-0.67	1,662,520	28,241	0.4821	120	0.44	78,350	217,163	0.3495	0.2813	0.7975
	90%	227	0.48	123	168,524	0.3611	160	273,229	0.2103	0.1859	134	-0.15	111,443	55,566	0.4925	134	0.61	82,432	166,004	0.3299	0.2876	0.8081
	Max	243	0.51	139	186,514	0.3316	180	292,501	0.1720	0.2034	145	0.10	-308,831	62,970	0.4954	144	0.61	85,198	131,342	0.3166	0.2919	0.8152

Outlier % = 95%

Laboratory		*Experimental Values*		*Modelled Values*																		
				k-theta			*Uzan*				*Boyce*					*Mayhew*						
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	υ_c	Ga	Ka	n	Mr_c	υ_c	Ga	Ka	n	β	m
UNOT	SFB	196	0.29	99	143,264	0.4245	95	226,306	0.4446	-0.0217	107	**-0.62**	179,631	25,595	0.5355	142	0.61	86,648	56,914	0.3184	0.1777	0.6415
LRSB(1)	SFB	207	0.22	89	135,557	0.4751	114	248,089	0.3555	0.1145	82	0.86	62,422	116,042	0.1353	107	0.23	86,130	63,502	0.3178	0.0467	0.3913
LRSB(2)	SFB	195	0.29	84	125,936	0.4590	118	246,300	0.3061	0.2089	61	**-1.00**	**728,384,849**	11,983	0.3947	114	0.26	76,468	89,795	0.3945	0.0405	0.4502
LRCF(1)	SFB	239	0.43	131	180,004	0.3628	174	302,660	0.2166	0.2019	139	0.17	95,188	75,087	0.4238	119	0.59	88,753	270,438	0.2261	0.5150	1.1090
LRCF(2)	SFB	153	0.51	93	123,836	0.3308	127	186,392	0.1466	0.1428	80	**-0.57**	125,651	20,006	0.5156	**-141**	**-2.84**	51,264	**-203,471**	0.5154	**-0.5424**	**-0.3376**
Values	Min	153	0.22	84	123,943	0.4362	95	213,602	0.3771	0.0612	61	-1.00	361,897,755	34,624	0.3588	-141	-2.84	51,310	-198,200	0.5150	-0.5439	-0.3435
	10%	170	0.24	86	126,391	0.4327	103	220,691	0.3563	0.0782	69	-0.85	311,524,427	38,148	0.3687	-42	-1.61	63,902	-77,872	0.4388	-0.2633	0.0334
	Avg	198	0.35	99	141,720	0.4104	126	241,949	0.2939	0.1293	94	-0.23	145,769,548	49,743	0.4010	68	-0.23	77,853	55,436	0.3544	0.0475	0.4509
	90%	226	0.48	118	164,122	0.3779	155	269,390	0.2133	0.1952	126	0.58	-67,949,174	64,693	0.4426	133	0.60	86,058	133,847	0.3048	0.2304	0.6965
	Max	239	0.51	131	179,417	0.3557	174	286,588	0.1628	0.2365	139	0.86	-151,791,036	70,558	0.4590	142	0.61	87,198	144,739	0.2979	0.2558	0.7306

Table continued…………..

Outlier % = 90%

Laboratory		*Experimental Values*		*Modelled Values*																		
				k-theta			*Uzan*				*Boyce*					*Mayhew*						
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	υ_c	Ga	Ka	n	Mr_c	υ_c	Ga	Ka	n	β	m
UNOT	SFB	195	0.28	94	138,895	0.4496	92	226,629	0.4586	-0.0103	107	**-0.50**	146,765	28,191	0.5048	138	0.61	86,637	55,865	0.3046	0.1747	0.6277
LRSB(1)	SFB	210	0.21	101	145,952	0.4177	107	237,142	0.3903	0.0408	91	**-0.59**	175,013	22,948	0.4369	107	0.23	86,128	63,255	0.3178	0.0470	0.3897
LRSB(2)	SFB	199	0.28	94	136,552	0.4223	110	234,155	0.3460	0.1156	74	**-1.00**	**148,006,582**	14,240	0.4702	115	0.28	76,474	85,006	0.3945	0.0482	0.4262
LRCF(1)	SFB	239	0.44	122	175,826	0.4127	162	304,417	0.2619	0.1802	129	0.19	97,965	68,684	0.3609	112	0.56	89,820	292,134	0.2000	0.5546	1.3004
LRCF(2)	SFB	151	0.51	93	123,236	0.3168	127	182,590	0.1408	0.1374	90	**-0.32**	86,761	27,511	0.5244	1,376	**16.76**	50,053	62,183	0.5321	0.2230	0.1739
Values	Min	151	0.21	93	132,626	0.3989	92	210,399	0.4030	0.0229	74	-1.00	80,797,047	9,374	0.5057	107	0.23	85,060	124,914	0.3022	0.2061	0.6905
	10%	169	0.24	93	132,791	0.3990	98	216,113	0.3850	0.0379	80	-0.84	67,327,281	15,422	0.4935	109	0.25	85,002	124,809	0.3026	0.2061	0.6897
	Avg	199	0.34	101	144,092	0.4038	119	236,986	0.3195	0.0927	98	-0.45	29,702,617	32,315	0.4595	370	3.69	77,822	111,689	0.3498	0.2095	0.5836
	90%	227	0.48	114	163,588	0.4121	148	264,538	0.2330	0.1652	120	-0.01	-16,124,569	52,890	0.4179	881	10.30	63,733	85,943	0.4424	0.2162	0.3753
	Max	239	0.51	122	176,379	0.4176	162	278,195	0.1902	0.2010	129	0.19	-34,381,473	61,087	0.4014	1,376	16.76	50,080	60,995	0.5322	0.2227	0.1735

Outlier % = 85%

Laboratory		*Experimental Values*		*Modelled Values*																		
				k-theta			*Uzan*				*Boyce*					*Mayhew*						
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	υ_c	Ga	Ka	n	Mr_c	υ_c	Ga	Ka	n	β	m
UNOT	SFB	193	0.28	92	137,160	0.4578	93	227,727	0.4492	0.0102	114	**-0.29**	113,967	36,195	0.4872	137	0.61	86,450	55,833	0.3017	0.1747	0.6278
LRSB(1)	SFB	208	0.21	97	142,806	0.4365	104	238,039	0.4043	0.0490	92	**-0.44**	137,310	25,932	0.4005	104	0.24	86,617	62,669	0.2971	0.0484	0.3856
LRSB(2)	SFB	201	0.28	101	142,831	0.3978	103	225,244	0.3842	0.0295	81	**-1.00**	**310,359,731**	15,348	0.5112	115	0.30	77,430	79,336	0.3782	0.0561	0.3989
LRCF(1)	SFB	241	0.44	125	178,681	0.4047	167	306,170	0.2493	0.1822	136	0.28	94,454	84,600	0.3718	135	0.57	90,604	285,344	0.2847	0.5104	1.2643
LRCF(2)	SFB	149	0.51	93	122,155	0.3098	124	179,279	0.1478	0.1262	100	**-0.02**	66,721	42,083	0.5269	614	**7.03**	49,118	97,891	0.5352	0.4365	0.3480
Values	Min	149	0.21	92	130,475	0.3992	93	210,299	0.3981	0.0201	81	-1.00	156,991,890	13,404	0.5025	104	0.24	86,514	120,149	0.3079	0.1844	0.6724
	10%	167	0.24	92	131,235	0.3993	97	214,435	0.3863	0.0299	86	-0.77	139,148,180	18,564	0.4944	109	0.26	86,210	120,008	0.3098	0.1865	0.6700
	Avg	199	0.34	102	144,726	0.4013	118	235,292	0.3269	0.0794	105	-0.29	62,154,437	40,832	0.4595	221	1.75	78,044	116,215	0.3594	0.2452	0.6049
	90%	228	0.48	115	164,668	0.4042	149	266,586	0.2379	0.1538	127	0.16	-29,144,178	67,236	0.4181	423	4.46	63,348	109,389	0.4487	0.3508	0.4877
	Max	241	0.51	125	178,834	0.4063	167	283,793	0.1889	0.1946	136	0.28	-64,574,827	77,483	0.4021	614	7.03	49,466	102,940	0.5331	0.4505	0.3771

Outlier % = 80%

Laboratory		*Experimental Values*		*Modelled Values*																		
				k-theta			*Uzan*				*Boyce*					*Mayhew*						
		Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	υ_c	Ga	Ka	n	Mr_c	υ_c	Ga	Ka	n	β	m
UNOT	SFB	193	0.27	89	134,938	0.4695	90	226,651	0.4639	0.0068	108	**-0.37**	122,161	32,180	0.4805	137	0.61	85,760	55,247	0.3078	0.1731	0.6221
LRSB(1)	SFB	205	0.21	95	140,691	0.4421	102	236,678	0.4070	0.0543	128	0.48	73,355	126,341	0.3951	105	0.25	85,910	62,184	0.3006	0.0500	0.3824
LRSB(2)	SFB	198	0.28	99	140,377	0.4004	100	220,168	0.3931	0.0159	131	0.69	64,743	312,439	0.4041	117	0.31	76,007	78,129	0.3928	0.0593	0.3926
LRCF(1)	SFB	243	0.44	129	182,135	0.3944	165	305,968	0.2590	0.1694	132	0.09	104,402	62,720	0.3891	134	0.58	90,962	274,198	0.2762	0.4928	1.2026
LRCF(2)	SFB	149	0.51	94	122,540	0.3034	123	178,132	0.1504	0.1218	102	0.03	64,387	46,148	0.5331	334	**3.34**	48,811	124,479	0.5406	0.6475	0.5443
Values	Min	149	0.21	89	128,055	0.4081	90	207,776	0.4090	0.0159	102	-0.37	89,453	28,260	0.5204	105	0.25	87,396	113,467	0.3025	0.1462	0.6386
	10%	167	0.24	91	130,513	0.4072	94	211,774	0.3974	0.0248	105	-0.21	88,986	39,490	0.5101	110	0.27	86,604	113,897	0.3074	0.1572	0.6378
	Avg	198	0.34	101	144,136	0.4020	116	233,519	0.3347	0.0736	120	0.18	85,810	115,966	0.4404	165	1.02	77,490	118,847	0.3636	0.2845	0.6288
	90%	228	0.48	117	165,137	0.3939	148	265,319	0.2429	0.1450	132	0.60	83,496	171,673	0.3896	255	2.25	62,763	126,847	0.4544	0.4902	0.6142
	Max	243	0.51	129	181,376	0.3877	165	282,171	0.1943	0.1828	132	0.69	83,404	173,869	0.3876	334	3.34	49,878	133,845	0.5338	0.6701	0.6015

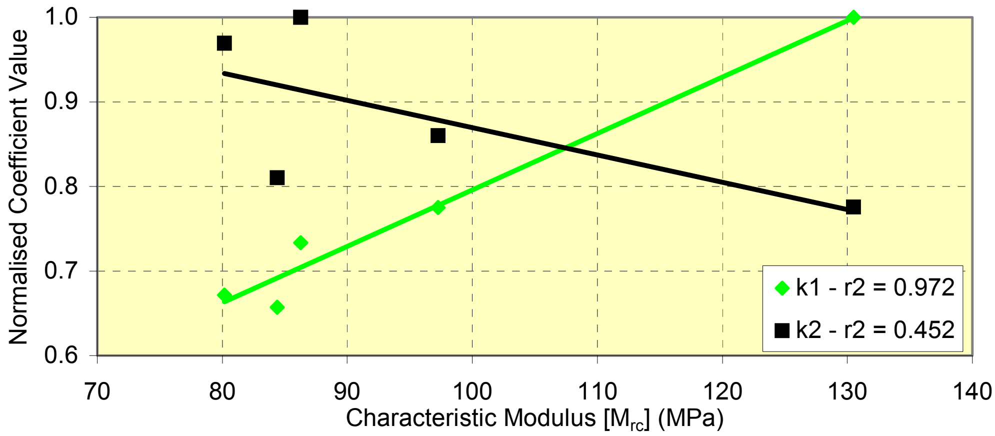
For each material's test results at the particular set of results that the outliers have been removed from, a summary is made. This is shown in Table 8-5 and Table 8-6 in white. A linear regression has then been conducted with these material parameters or model coefficients against the characteristic resilient modulus for a subgrade soil and an unbound base material as shown diagrammatically in Figure 8-3, where the model coefficient value is normalised such that its maximum value is 1. The results of this analysis are tabulated and called *Values,* comprising five rows in Table 8-5 and Table 8-6, in green, and are as follows:

- Min Is the value of the material parameters or model coefficients corresponding to the minimum value of the resilient modulus on the regression line
- 10% Is the value of the material parameters or model coefficients corresponding to the 10^{th} percentile value of the resilient modulus on the regression line
- Avg Is the value of the material parameters or model coefficients corresponding to the average value of the resilient modulus on the regression line resulting parameters or model coefficients
- 90% Is the value of the material parameters or model coefficients corresponding to the 90^{th} percentile value of the resilient modulus on the regression line
- Max Is the value of the material parameters or model coefficients corresponding to the maximum value of the resilient modulus on the regression line

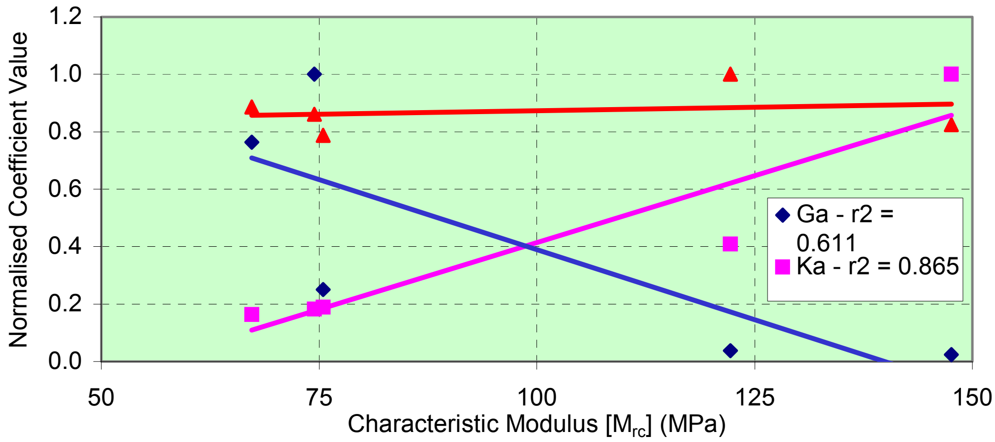
This analysis produces a range of values of parameters and coefficients from each analysis; the effect of the difference in magnitude within this range on the design of pavements is investigated in the next chapter.

Figure 8-3 Material Coefficient as a Percentile of Resilient Modulus for Fontainebleau Sand tested in Test Programme I

k-theta Model Subgrade Soil



Boyce Model Unbound Granular Material



Note: All Coefficients have been normalised by dividing by the maximum value.

8.3.4 Comparison of the Results as the Data is Reduced by Removal of Outliers

The correlation coefficient has been calculated for the analysis of all results on all specimens tested. By using the correlation coefficient it is possible to define an optimum percentile value at which the outliers should be excluded from test data. Two examples of the comparison between the experimental and the modelled resilient modulus for a subgrade soil and an unbound granular material are shown in Figure 8-4 and Figure 8-5.

Figure 8-4 Comparison for all Stress Paths showing Probable outliers for a Specimen of Fontainebleau Sand

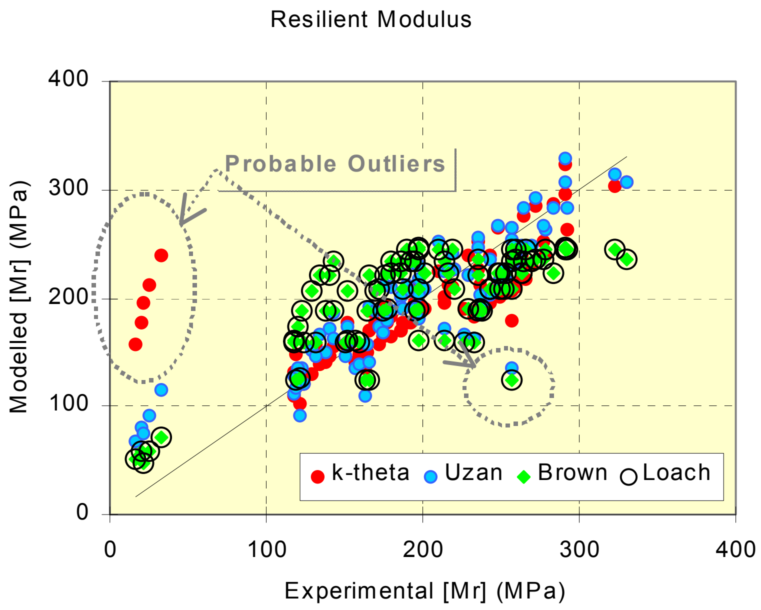


Figure 8-5 Comparison for all Stress Paths showing Probable outliers for a Specimen of Hard Limestone

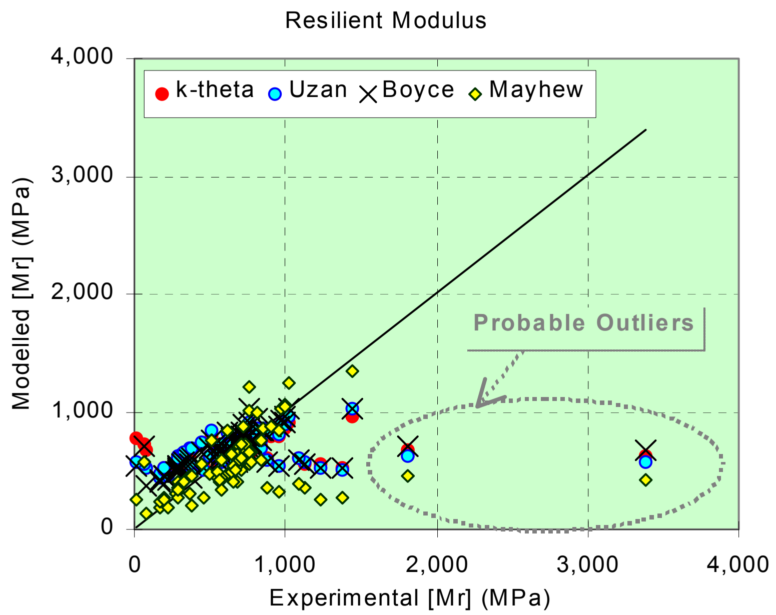


Figure 8-6 Results from a Specimen of Fontainebleau Sand once the 90% Outliers have been Removed

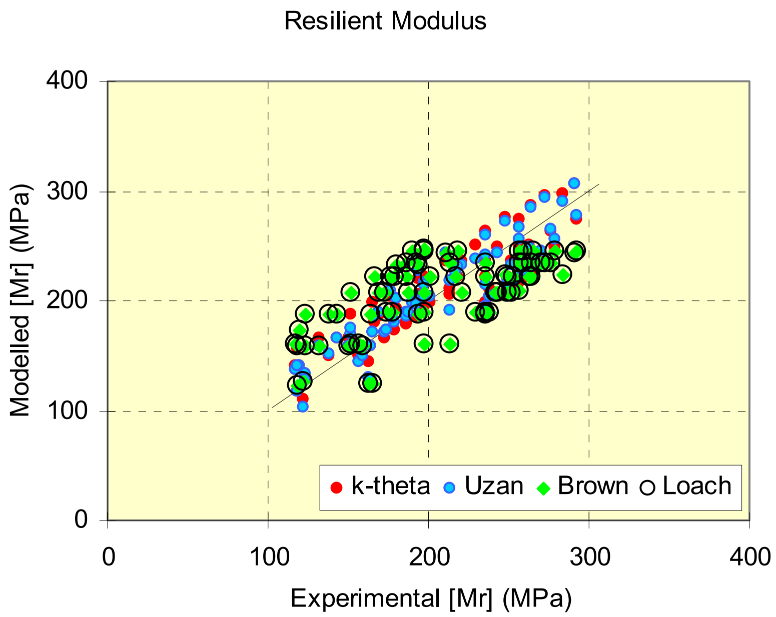
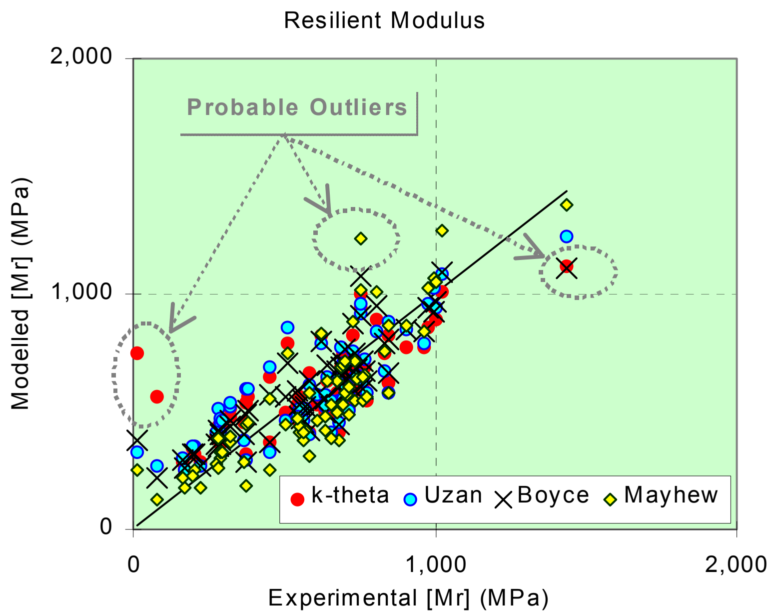


Figure 8-7 Results from a Specimen of Hard Limestone once the 90% Outliers have been Removed



The examples shown in Figure 8-4 and Figure 8-5 contain all of the test data i.e. 100th percentile data. Clearly there are some results that do not appear to conform to the model resulting in a poor correlation value. These are marked as 'Probable outliers'. Figure 8-6 and Figure 8-7 show the same data once the outliers have been removed, resulting in a much neater fit for the sand. The limestone, however, may still show some outliers. Some comment is necessary about the way these plots show the results and whether the apparent outliers are really outliers. It must be remembered that each point in these figures represents a single stress path, and outliers are defined as the absolute difference between the experimental and modelled values for resilient modulus for a single stress path. If a difference is determined for one model that defines the point as an outlier then that point (data for the particular stress path) is removed. Therefore the point is removed for all of the models, and in the examples above four points are removed from the graph (one for each model) which may give the impression that some points that were not outliers were removed. For example the points in Figure 8-4 that are around the 20 MPa experimental value and the 50 MPa modelled value correlate well for the Uzan, Brown and Loach models but are results from the same stress path as those red points above them (k-theta model) which are marked as probable outliers, thus when the stress path is removed from the data (Figure 8-5) all of the points are removed. The goodness of the fit is measured by comparing the correlation coefficients for each model as shown in Table 8-7.

Table 8-7 Correlation Coefficients at Various Outlier Removal Percentile Values for a Specimen of Fontainebleau Sand and Hard Limestone

Material	**Correlation Coefficient (r^2) for Models**				
SFB	**k-theta**	**Uzan**	**Brown**	**Loach**	
%'ile	**(Mr)**	**(Mr)**	**(Mr)**	**(Mr)**	
100%	0.272	0.801	0.667	0.667	
95%	0.394	0.799	0.623	0.623	
90%	0.669	0.784	0.478	0.478	
85%	0.705	0.830	0.504	0.504	
80%	0.828	0.850	0.371	0.371	
CCD	**k-theta**	**Uzan**	**Boyce**	**Mayhew**	
%'ile	**(Mr)**	**(Mr)**	**(Mr)**	**(ε_v)**	**(ε_s)**
100%	0.316	0.321	0.351	0.694	0.646
95%	0.447	0.525	0.547	0.689	0.648
90%	0.529	0.598	0.662	0.696	0.656
85%	0.615	0.636	0.717	0.757	0.675
80%	0.674	0.674	0.733	0.744	0.674

For these two specimens, Fontainebleau Sand and Hard Limestone, the correlation coefficient for the k-theta model becomes considerably better with the removal of the outliers. The Uzan model shows a good fit for the subgrade soil specimen and this fit changes little when the outliers are removed. This is true for the Mayhew model for unbound granular materials as well. When the Uzan and Boyce models are applied to the specimen of Hard Limestone, a poor correlation is found initially but this improves greatly with the removal of the outliers.

Since the data is always the same for all models, if a model was found to have a bad correlation for a particular data set (stress path) and the other three models were found to have a good correlation for the same set of data the data is removed from all models. This should result in a correlation value becoming higher, or better, for one model however this may affect other models in different ways and it is possible that the correlation in another model remains constant or even becomes worse. As shown in these examples, the decline of the correlation with the removal of outliers did not always occur during the analysis.

There exist three possible trends in the correlation coefficient with the removal of outliers:

- The correlation improves.
- There is little change.
- The correlation deteriorates.

A summary of the three possible trends is shown in Table 8-8 for all of the specimens conducted in Test Programme I.

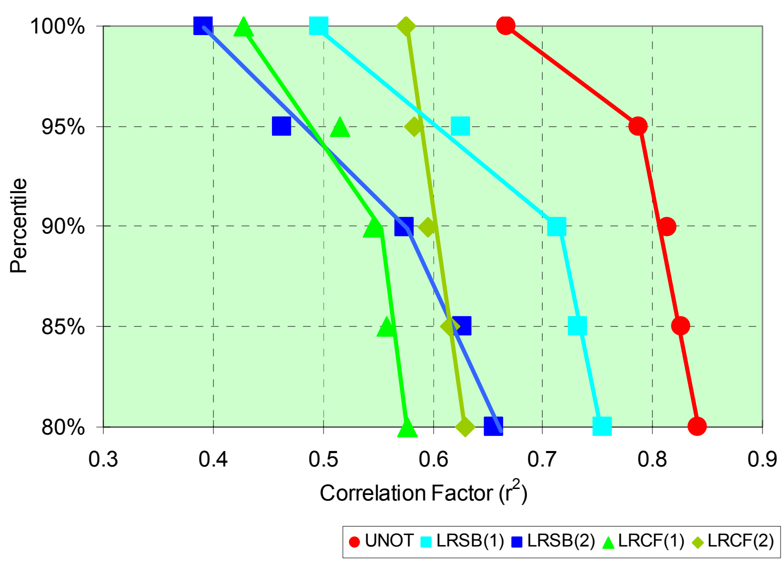
Table 8-8 Summary of the Trends of the Correlation Coefficients for the Removal of Outliers for Test Programme I

Trend Summary	**Subgrade Soils**			**Unbound Granular Material**				
	SFB	LOC	**Total**	SFB	CCD	CCT	MIG	**Total**
Improving	14	7	**21**	12	3	3	3	**21**
No Change	4	4	**8**	13	12	7	2	**34**
Deteriorating	2	1	**3**	0	0	0	0	**0**
Total	**20**	**12**	**32**	**25**	**15**	**10**	**5**	**55**

Importantly, only 3 of the total of 87 analyses resulted in the correlation coefficient deteriorating and is, therefore, negligible. Of those in which there was no change, 87% started with a correlation coefficient of greater than 0.5 while 58% started with a correlation coefficient of greater than 0.7. Of those results where the correlation coefficients were found to improve only 51% started with a correlation coefficient of greater than 0.5 and 40% started with a correlation coefficient of greater than 0.7.

Looking at all of the results from all of the tests conducted in Test Programme I it was found that there is approximately an improvement in the correlation at 90th percentile but little improvement there after. This is illustrated in the example shown in Figure 8-8. The correlation factor for the LRSB(1), LRSB(2) and LRCF(1) data improves more rapidly between 100% and 90% than between 90% and 80%. It is observed that this is not true for all of the data, the UNOT data improved rapidly from 100% to 95% and then improves slowly to 80% at about the same rate as the other data. Whereas the LRCF(2) data improve at the same rate with no rapid improvement.

Figure 8-8 Correlation factors for Differing Percentile Values for Fontainebleau Sand Modelled using the k-theta Model



Graphs and the values for all of the materials tested in Test Programme I are shown in Appendix G.4.

Conclusions from the Removal of Outliers

Based on the summary of the correlation coefficients from Test Programme I as above it is concluded that:

- The effect of removing outliers on the correlation coefficients shows that generally there is an improvement in the modelled data.
- Those tests where the correlation starts off higher, benefit less from the removal of outliers than those with poor initial correlation coefficients.
- Little improvement in the correlation coefficient is achieved when more than the 90^{th} percentile data are removed.

8.3.5 Comparison of Identical Data Analysed using Different Analytical Methods

During Test Programme I Fontainebleau Sand was tested at three different laboratories, however, five specimens in total. This dry sand is a fine-grained material, which is thought to behave like an unbound granular material under repeated loading. Therefore this material was suitable to be analysed using the method developed for subgrade soils as well as the method for unbound granular materials. Thus, all seven analytical models used in this work were applied to the same sets of test results. The resulting characteristic resilient moduli for both the experimental data and the modelled data are shown in Table 8-9.

Table 8-9 Characteristic Resilient Modulus for Fontainebleau Sand Analysed by two Different Methods

Laboratory	Characteristic Resilient Modulus (MPa)									
	Experimental Values		k-theta		Uzan		Brown	Loach	Boyce	Mayhew
	Soil	UGM	Soil	UGM	Soil	UGM	Soil	Soil	UGM	UGM
UNOT	154	198	97	104	120	112	157	157	122	147
LRSB(1)	169	205	91	87	100	128	176	176	75	110
LRSB(2)	151	196	89	87	91	125	150	150	74	114
LRCF(1)	207	245	145	144	166	184	206	206	148	121
LRCF(2)	130	155	87	93	114	133	136	136	67	113

The difference in the experimental values between the subgrade soil (Soil) and the Unbound Granular Material (UGM) is due to the different characteristic stresses applied to the equation. This shows the stress dependency of the material since the UGM, which is higher in the pavement, has greater applied stresses and consequently a higher resilient modulus than the subgrade soil which is lower in the pavement structure. Similarly, it is expected that the modelled values for the material analysed as a subgrade soil should be consistently lower than those for an unbound granular material. This is true for the experimental values and the simpler k-theta and Uzan models but not so for the more complex models. As a result of this the following analysis does not combine the results obtained using the soil and UGM models.

Two comparisons can be made from these results namely:

- To compare the range of the resulting resilient moduli obtained from the different model analysis using the results from a single laboratory's data, and;
- To compare the resulting resilient moduli obtained from a single model analysis type (for example k-theta) using the results from all of the laboratory's data.

The average, and the range, of modelled resilient moduli can thus be compared against one another and the characteristic experimental values for both comparisons.

These two comparisons are shown graphically in Figure 8-9 and Figure 8-10. In the characteristic experimental resilient modulus for each specimen tested (different laboratories) is shown as a point. So too is the average modelled value, however this point has an error bar included so depict the range of all of the modelled values.

Figure 8-9 clearly shows that when the results are analysed using the subgrade soil models that the experimental value is approximately equal to the upper limit of the range of the characteristic values obtained from the models. For each specimen the experimental value for the UGM is greater than the experimental value for the soil. However, the reverse is true for the average resilient modulus when predicted using the models. Further, the values predicted using the soil models show a greater variation than those of the UGM models.

When the results from each specimen as analysed by a particular model are shown as an average and range in Figure 8-10 it can be seen that again the modelled values are lower than the experimental values.

Figure 8-9 Comparison of Fontainebleau Sand Results Analysed for Different Specimens (Laboratories)

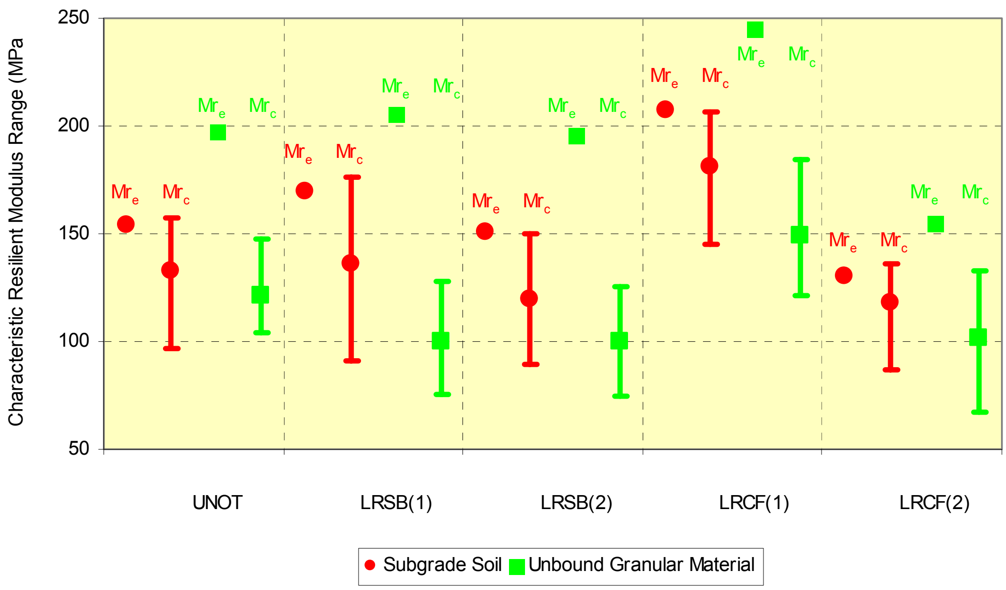
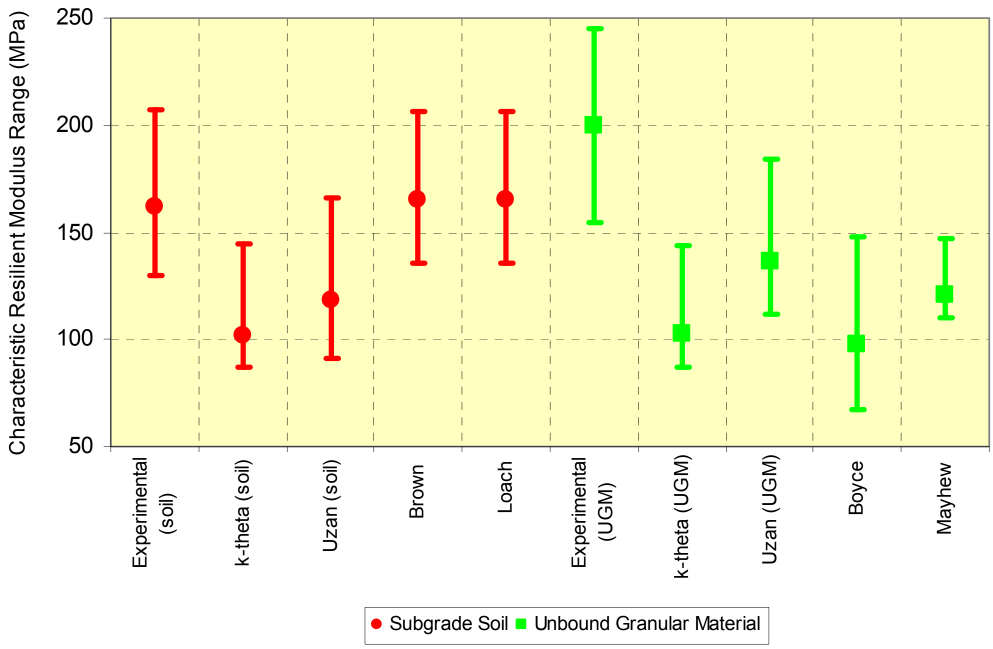


Figure 8-10 Comparison of Fontainebleau Sand Results Analysed by Different Analytical Methods



The k-theta models yield similar results for different levels of stress (i.e. when analysed as soil or as UGM), however Uzan varies somewhat with UGM yielding higher values than soils. Brown and Loach predict the highest values but these are similar to the experimental values. In general, the UGM models predict lower values than the soils. The variation of resilient modulus between the different specimens (laboratories) when analysed by the different methods for soils and UGM is between -21% and 44%. The variation of resilient modulus between the model methods for all specimens (laboratories) for soils and UGM and the average values are shown in Table 8-10.

Table 8-10 Variation of Resilient Moduli when Predicted by Different Methods of Modelling

Model	**Soil Models**			**UGM Models**			**Both Models**		
Average	137 MPa			114 MPa			126 MPa		
Variation	**Min**	**Max**	**Range**	**Min**	**Max**	**Range**	**Min**	**Max**	**Range**
Specimens	-20%	32%	**51%**	-22%	30%	**53%**	-21%	44%	**64%**
Models	-26%	20%	**46%**	-15%	19%	**34%**	-23%	31%	**54%**

Therefore greater variation occurs between the resilient moduli as predicted from analysing the results from a single specimen (laboratory) using different models than there is when the results from different specimens are analysed using a single model.

8.3.6 Comparison of the Same Material Tested at Different Laboratories

After considering the results from Test Programme I, a second test programme was formulated in order that the results from different laboratories testing similar materials could be compared. Two materials were tested at four laboratories, one subgrade soil – London Clay and the other an unbound granular material - Microgranite. The test procedure for the soil required few stress paths and therefore no stress paths (outliers) were excluded. However, there were substantially more stress paths applied to the unbound granular material specimens and an analysis was conducted for all of the data (100th percentile) and the 90th percentile data. Complete results of the analysis are contained in the tables below and Appendix G. These results are shown graphically in Figure 8-11 and Figure 8-12, which provide a graphic representation of the difference in the results from the various laboratories.

Figure 8-11 Comparison of the Analysis of London Clay (Test Programme II) tested at four Laboratories

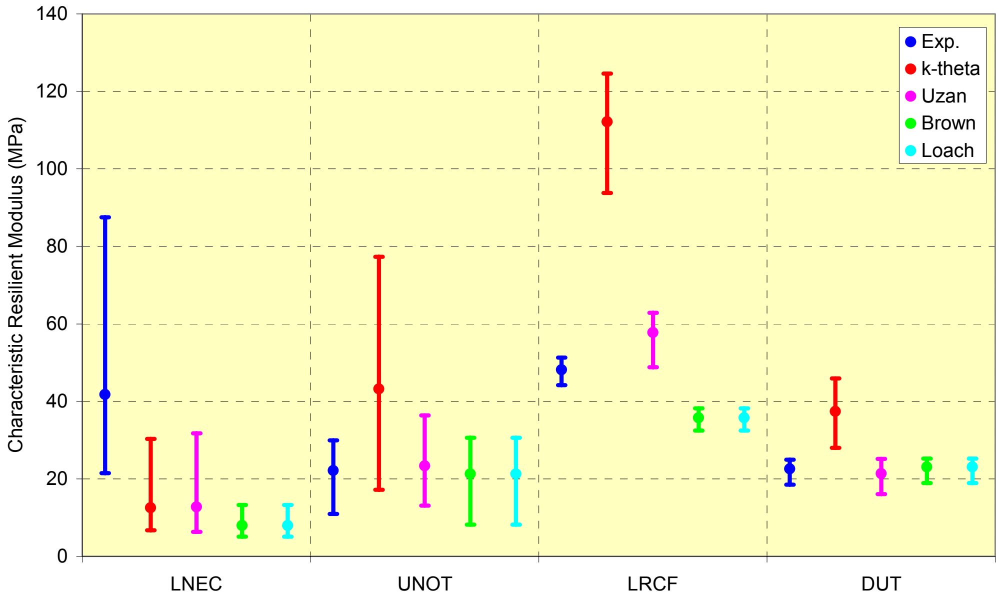
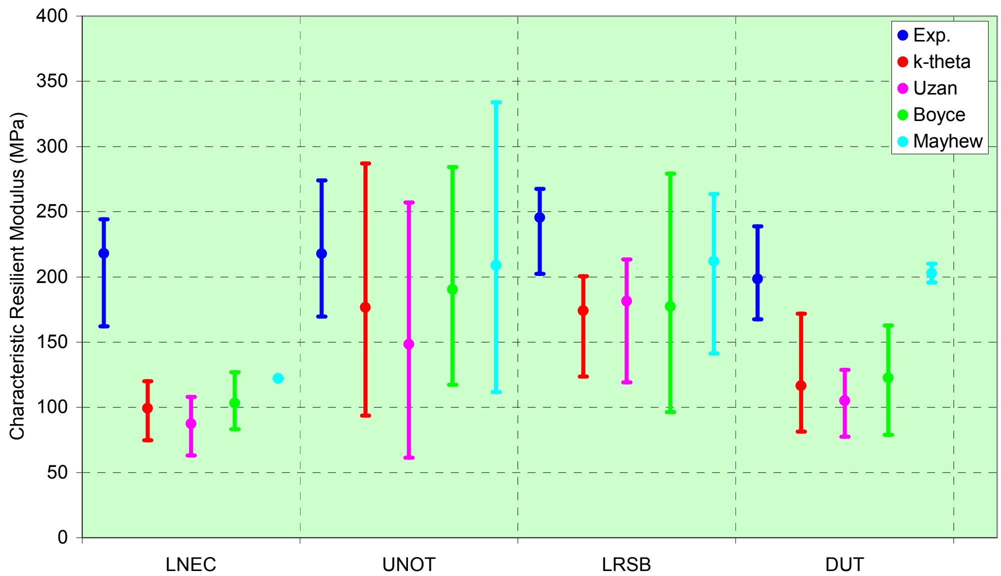


Figure 8-12 Comparison of the Analysis of Microgranite (Test Programme II) tested at four Laboratories



For the London Clay tested at LNEC, UNOT and DUT the characteristic modelled resilient modulus was found to be approximately between 10 and 30 MPa. LRCF, however, produced a considerably higher estimation of the resilient modulus for the k-theta model (110 MPa), excluding this result the range of predicted average resilient moduli from LRCF is between 35 and 60 MPa. The actual results of the analysis on London Clay is shown in Table 8-11 and summarised in Table 8-13.

Table 8-11 Results of Test Programme II on Subgrade Soil – London Clay

Laboratory	Experimental Values		Modelled Values (MPa)			
			k-theta	Uzan	Brown	Loach
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	Mr_c
LNEC	22	0.54	7	6	5	5
	24	0.67	9	9	6	6
	59	0.74	12	12	10	10
	28	0.64	9	9	7	7
	30	1.19	8	8	6	6
	88	3.13	30	32	13	13
Min	**22**	**0.54**	**7**	**6**	**5**	**5**
Avg	**42**	**1.15**	**13**	**13**	**8**	**8**
Max	**88**	**3.13**	**30**	**32**	**13**	**13**
UNOT	11	0.78	77	31	8	8
	23	0.57	46	17	18	18
	29	0.81	55	27	29	29
	19	0.39	17	13	20	20
	30	0.42	43	36	31	31
	21	1.10	21	16	22	22
Min	**11**	**0.39**	**17**	**13**	**8**	**8**
Avg	**22**	**0.68**	**43**	**23**	**21**	**21**
Max	**30**	**1.10**	**77**	**36**	**31**	**31**
LRCF	51	0.76	118	62	38	38
	49	0.86	125	63	37	37
	44	0.60	94	49	32	32
Min	**44**	**0.60**	**94**	**49**	**32**	**32**
Avg	**48**	**0.74**	**112**	**58**	**36**	**36**
Max	**51**	**0.86**	**125**	**63**	**38**	**38**
DUT	18	0.37	28	16	19	19
	24	0.39	38	23	25	25
	25	0.49	46	25	25	25
Min	**18**	**0.37**	**28**	**16**	**19**	**19**
Avg	**23**	**0.42**	**37**	**21**	**23**	**23**
Max	**25**	**0.49**	**46**	**25**	**25**	**25**

Table 8-12 Results of Test Programme II on Unbound Granular Material - Microgranite

Laboratory	Experimental Values		Modelled Values (MPa)					
			k-theta	Uzan	Boyce		Mayhew	
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	υ_c	Mr_c	υ_c
LNEC	162	0.2	85	78	88	-0.4		
	194	0.4	75	63	83	-0.1		
	244	0.1	120	104	107	-0.7		
	244	0.2	92	83	127	0.9		
	238	0.2	115	108	111	-0.3	122	0.4
	225	0.1	108	89	102	-0.4		
Min	**162**	**0.1**	**75**	**63**	**83**	**-0.7**	**122**	**0.4**
Avg	**218**	**0.2**	**99**	**87**	**103**	**-0.2**	**122**	**0.4**
Max	**244**	**0.4**	**120**	**108**	**127**	**0.9**	**122**	**0.4**
UNOT	169	0.2	94	61	117	0.5	112	0.4
	274	0.4	287	257	284	-1.0	334	0.5
	210	0.5	149	126	170	-0.2	181	0.5
Min	**169**	**0.2**	**94**	**61**	**117**	**-1.0**	**112**	**0.4**
Avg	**218**	**0.4**	**177**	**148**	**190**	**-0.2**	**209**	**0.5**
Max	**274**	**0.5**	**287**	**257**	**284**	**0.5**	**334**	**0.5**
LRSB	266	0.3	200	213	279	0.0	231	-0.1
	268	0.4	198	212	156	-1.0	263	0.4
	202	0.4	123	119	96	-0.8	141	0.4
Min	**202**	**0.3**	**123**	**119**	**96**	**-1.0**	**141**	**-0.1**
Avg	**245**	**0.3**	**174**	**181**	**177**	**-0.6**	**212**	**0.2**
Max	**268**	**0.4**	**200**	**213**	**279**	**0.0**	**263**	**0.4**
DUT	239	0.2	172	129	163	-0.9		
	189	0.2	96	109	126	-0.4	210	0.1
	167	0.2	81	77	79	-0.8	196	0.7
Min	**167**	**0.2**	**81**	**77**	**79**	**-0.9**	**196**	**0.1**
Avg	**198**	**0.2**	**116**	**105**	**123**	**-0.7**	**203**	**0.4**
Max	**239**	**0.2**	**172**	**129**	**163**	**-0.4**	**210**	**0.7**

Table 8-13 Summary of the Test Programme II Subgrade Soil Results

Laboratory	Experimental Values		Modelled Values (MPa)			
			k-theta	Uzan	Brown	Loach
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	Mr_c
LNEC	42	1.15	13	13	8	8
UNOT	22	0.68	43	23	21	21
LRCF	48	0.74	112	58	36	36
DUT	23	0.42	37	21	23	23

For the Microgranite LNEC and DUT estimate the resilient modulus to be between 60 and 250 MPa whereas the other two laboratories produced a higher estimation of up to approximately 340 MPa. The actual results of the analysis on Microgranite are shown in Table 8-12 and summarised in Table 8-14.

Table 8-14 Summary of the Test Programme II Unbound Granular Material – Microgranite Results

Laboratory	Experimental Values		Modelled Values (MPa)					
			k-theta	Uzan	Boyce		Mayhew	
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	υ_c	Mr_c	υ_c
LNEC	218	0.2	99	87	103	-0.2	122	0.4
UNOT	218	0.4	177	148	190	-0.2	209	0.5
LRSB	245	0.3	174	181	177	-0.6	212	0.2
DUT	198	0.2	116	105	123	-0.7	203	0.4

8.3.7 Comparison of Different Specimens of the Same Material Tested within a Single Laboratory

In test Programme III, two subgrade soil materials were tested extensively at LNEC and two unbound granular materials were tested at LRSB. The materials tested were as follows:

Laboratory	Material	No. of Specimens
LNEC	London Clay (LOC)	12
	Seine at Marne (LIM)	12
LRSB	Soft Limestone (CCD)	4
	Hard Limestone (CCT)	4

The test results from this programme are best illustrated graphically as shown in Figure 8-13 to Figure 8-16, although the actual results from these tests are summarised in Table 8-16 to Table 8-19. The first observation is that the Brown and the Loach models give much the same characteristic values throughout this is because the models are fundamentally the same except that one considers the suction in the material, however for this work the suction is a constant and therefore both models provide the same result. For both soils, the characteristic experimental values were found to be lower than the characteristic modelled values. However, the opposite was found for the unbound granular materials.

The range of the experimental values for the soils is less than the range of the values estimated by the models for these materials however the range of the experimental values for the unbound granular materials is approximately equal to or greater than the modelled values. This implies that is not as important to use sophisticated models for lower layers (subgrade soils) as it is for the upper layers, in a pavement, where unbound granular materials are commonly used.

If the characteristic values resulting from different analytical methods for a particular specimen are averaged and the coefficient of variation calculated, it was always found to be higher that the variation from the averaged characteristic values from individual models. This is shown clearly in Table 8-15, and leads to the conclusion that selection of the most appropriate model will yield more accurate results and the variation in results from different laboratories will not be less significant.

Table 8-15 Variation from the Average for Average Modelled and Specimen Characteristic Values

Material	Coefficient of Variation from the Average Characteristic Resilient Modulus			
	Averaged Modelled		Averaged Specimen	
	Minimum	Maximum	Minimum	Maximum
London Clay (LOC)	-8%	19%	-82%	88%
Seine et Marne Silt (LIM)	-10%	12%	-51%	78%
Soft Limestone (CCT)	-20%	48%	-83%	115%
Hard Limestone (CCD)	-15%	22%	-78%	170%

Table 8-16 Results of Test Programme III on Subgrade Soil – London Clay

Material	Experimental Values		Modelled Values (MPa)			
			k-theta	Uzan	Brown	Loach
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	Mr_c
LOC	232	0.26		447	401	401
	180	0.33		352	241	241
	108	0.31	275	186	154	154
	84	0.26	153	109	100	100
	124	0.26	232	167	145	145
	95	0.30	148	122	105	105
	199	0.50		318	252	252
	136	0.49		255	175	175
	328	0.53			416	416
	93	0.22	33		44	44
	204	0.26		506	246	246
	153	0.21	496	164	167	167
Min	**84**	**0.21**	**33**	**109**	**44**	**44**
10%	**93**	**0.22**	**91**	**121**	**100**	**100**
Avg	**161**	**0.33**	**223**	**263**	**204**	**204**
90%	**229**	**0.50**	**385**	**453**	**386**	**386**
Max	**328**	**0.53**	**496**	**506**	**416**	**416**

Figure 8-13 Analysis of the London Clay Specimens tested at LNEC under Test Programme III

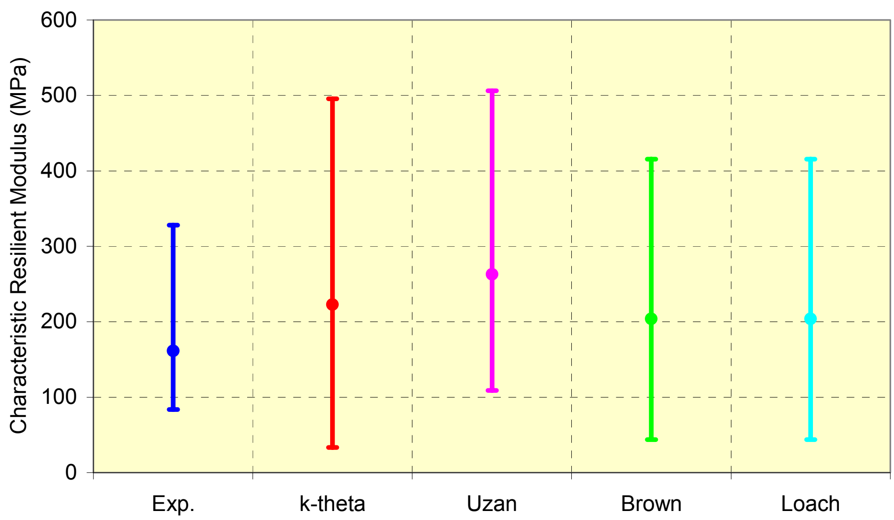


Table 8-17 Results of Test Programme III on Subgrade Soil – Seine et Marne Silt

Material	Experimental Values		Modelled Values (MPa)			
			k-theta	Uzan	Brown	Loach
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	Mr_c
LIM	213	0.24	245	175	220	220
	217	0.22	204	189	214	214
	195	0.24	284	185	224	224
	134	0.20	103	91	138	138
	132	0.52	244	253	141	141
	216	0.42	141	137	252	252
	144	0.34	102	108	154	154
	398	0.69	409	319	483	483
	184	0.22	345	245	238	238
	203	0.27	336	297	237	237
	269	0.22	470	321	334	334
	152	0.27	296	247	189	189
Min	**132**	**0.20**	**102**	**91**	**138**	**138**
10%	**135**	**0.22**	**107**	**111**	**143**	**143**
Avg	**205**	**0.32**	**265**	**214**	**235**	**235**
90%	**264**	**0.51**	**402**	**317**	**326**	**326**
Max	**398**	**0.69**	**470**	**321**	**483**	**483**

Figure 8-14 Analysis of the Seine et Marne Specimens Tested at LNEC under Test Programme III

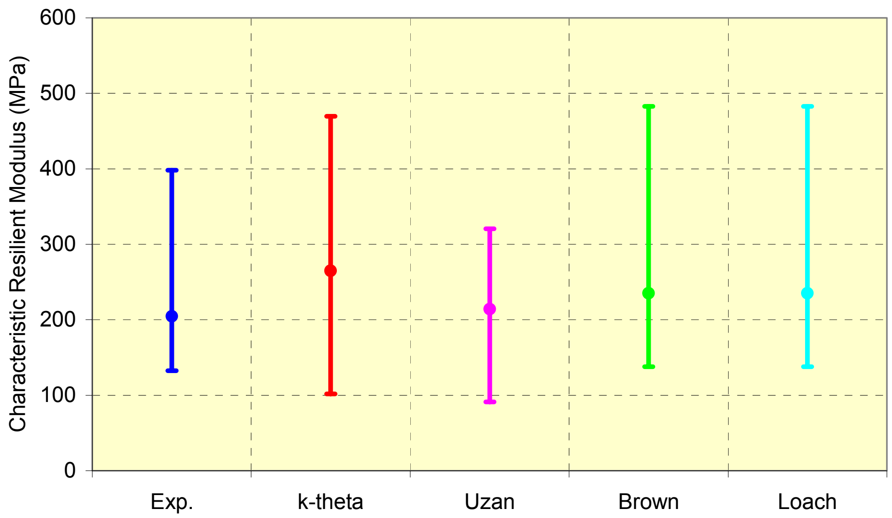


Table 8-18 Results of Test Programme III on Unbound Granular Material – Soft Limestone Results

Laboratory	Experimental Values		Modelled Values (MPa)					
			k-theta	Uzan	Boyce		Mayhew	
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	υ_c	Mr_c	υ_c
CCT	317	0.4	74	68	114	-0.7	90	0.3
	1413	0.3	817	609	530	-0.9	1100	0.2
	277	0.5	74	64	112	-0.7	105	0.5
	1089	0.4	820	963	873	-0.8	1696	0.3
Min	**277**	**0.3**	**74**	**64**	**112**	**-0.9**	**90**	**0.2**
10%	**289**	**0.3**	**74**	**65**	**113**	**-0.9**	**94**	**0.2**
Avg	**774**	**0.4**	**446**	**426**	**407**	**-0.8**	**748**	**0.3**
90%	**1316**	**0.5**	**819**	**857**	**770**	**-0.7**	**1517**	**0.4**
Max	**1413**	**0.5**	**820**	**963**	**873**	**-0.7**	**1696**	**0.5**

Figure 8-15 Analysis of the Soft Limestone Specimens Tested at LRSB under Test Programme III

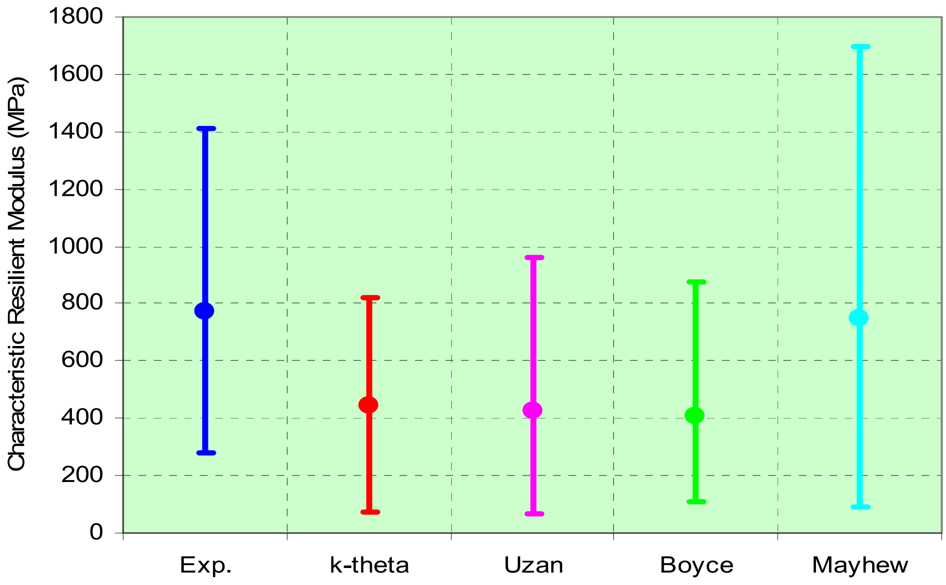
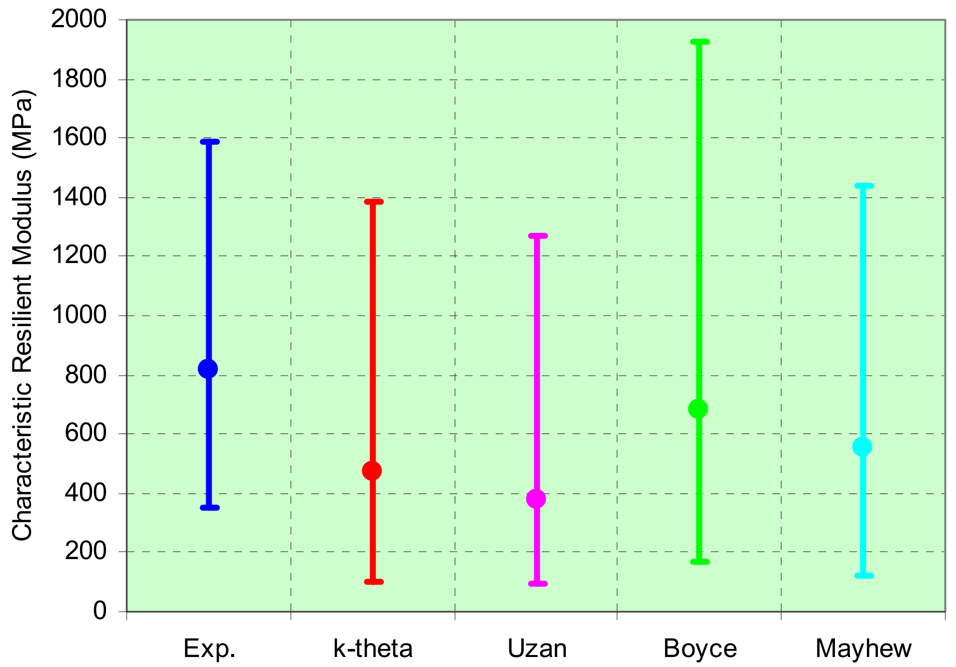


Table 8-19 Results of Test Programme III on Unbound Granular Material – Hard Limestone Results

Laboratory	Experimental Values		Modelled Values (MPa)					
			k-theta	Uzan	Boyce		Mayhew	
	Mr_e	υ_e	Mr_c	Mr_c	Mr_c	υ_c	Mr_c	υ_c
CCD	1586	0.6	1388	1273	1929	-0.1	1436	0.2
	536	0.3	177	157	344	0.2	305	0.2
	354	0.7	100	94	170	0.5	124	0.5
	794	0.2	238		285	-0.6	341	0.1
Min	**354**	**0.2**	**100**	**94**	**170**	**-0.6**	**124**	**0.1**
10%	**409**	**0.3**	**124**	**107**	**205**	**-0.4**	**178**	**0.1**
Avg	**817**	**0.5**	**476**	**381**	**682**	**0.0**	**552**	**0.2**
90%	**1348**	**0.7**	**1043**	**938**	**1453**	**0.4**	**1108**	**0.4**
Max	**1586**	**0.7**	**1388**	**1273**	**1929**	**0.5**	**1436**	**0.5**

Figure 8-16 Analysis of the Hard Limestone Specimens Tested at LRSB under Test Programme III



8.4 INTRODUCTION OF RANDOM ERRORS TO DATA

In order to test the influence that errors in the readings from the instrumentation have on the modelling and pavement design, a controlled error was introduced into what was otherwise 'perfect data'.

Two materials, an unbound granular material (CCT) and a subgrade soil (LOC) were selected and the strains calculated using a simple model as follows:

$$\varepsilon_{1a} = (a \times q_2 + b) + (a \times q_2 + b) \times F_{r1} \times V$$

$$\varepsilon_{3r} = (\varepsilon_{1a} \times v) + (\varepsilon_{1a} \times v \times F_{r3})$$

Eqn.8-2

Where Fr Random number between -1 and 1 (subscript indicated axial and radial)

V Variation

a and b Constants dependent on actual measurements

v Poisson's ratio (assumed constant) = 0.40

The coefficients for the models were calculated, as before, for different values of variation, (0%; 2%; 5%; 10%; 30%; 50%), for both the CCT and the LOC. Figure 8-17 and Figure 8-18 show the scatter of the points increasing as the variation increases for the two examples.

It was found that the introduction of a random error and the subsequent increase in the variation had very little effect on the material coefficients for variation of up to 30% for both materials and thus modelling methods. There was a 'jump' in the magnitude of the parameters and coefficients at 50% variation, however. All of the results of this model analysis can be found in Appendix G Also summary tables containing the material parameters and coefficients are contained in this appendix.

Figure 8-17 Increase in Scatter as the Variation Increases for a Subgrade Soil

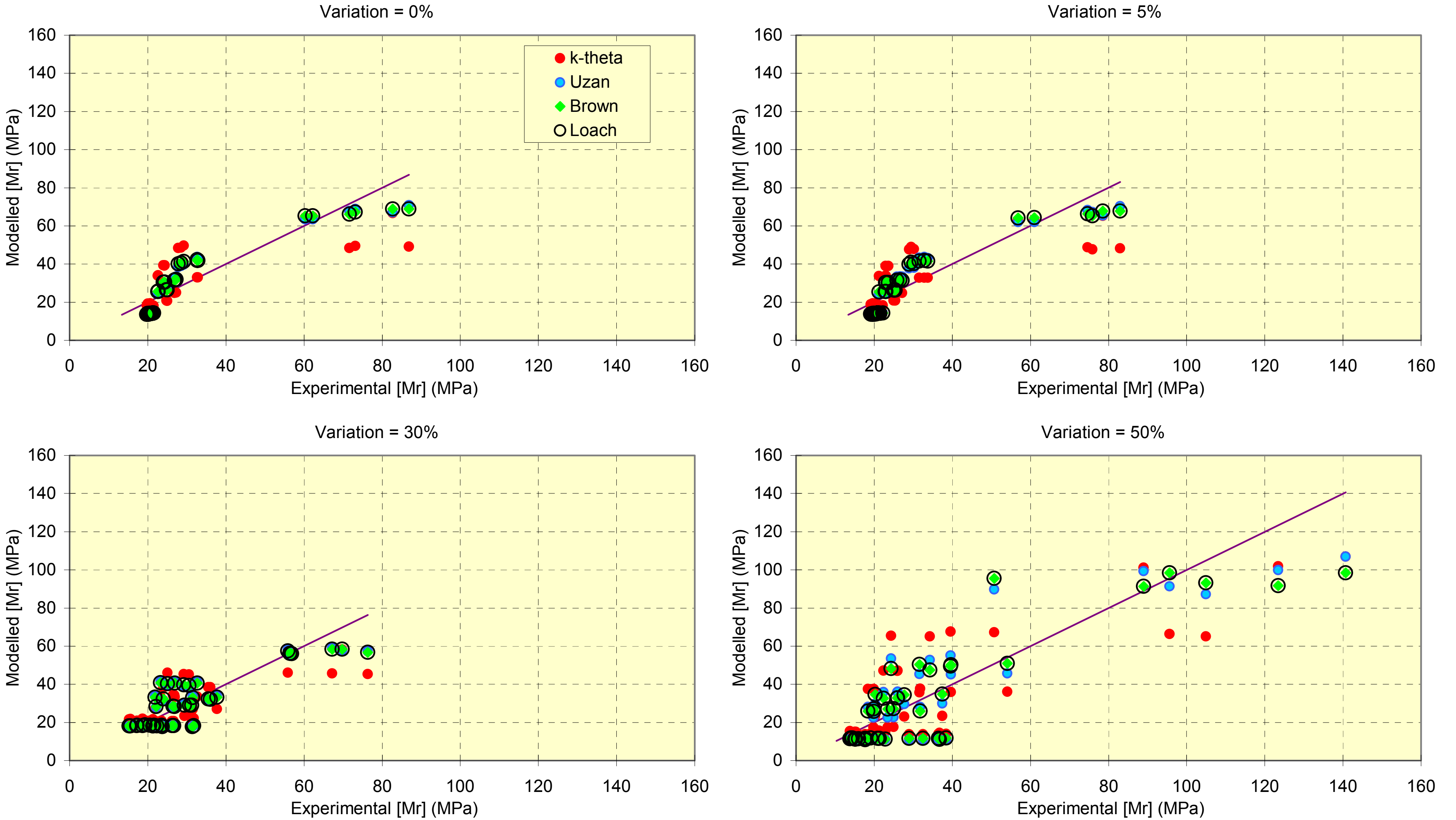
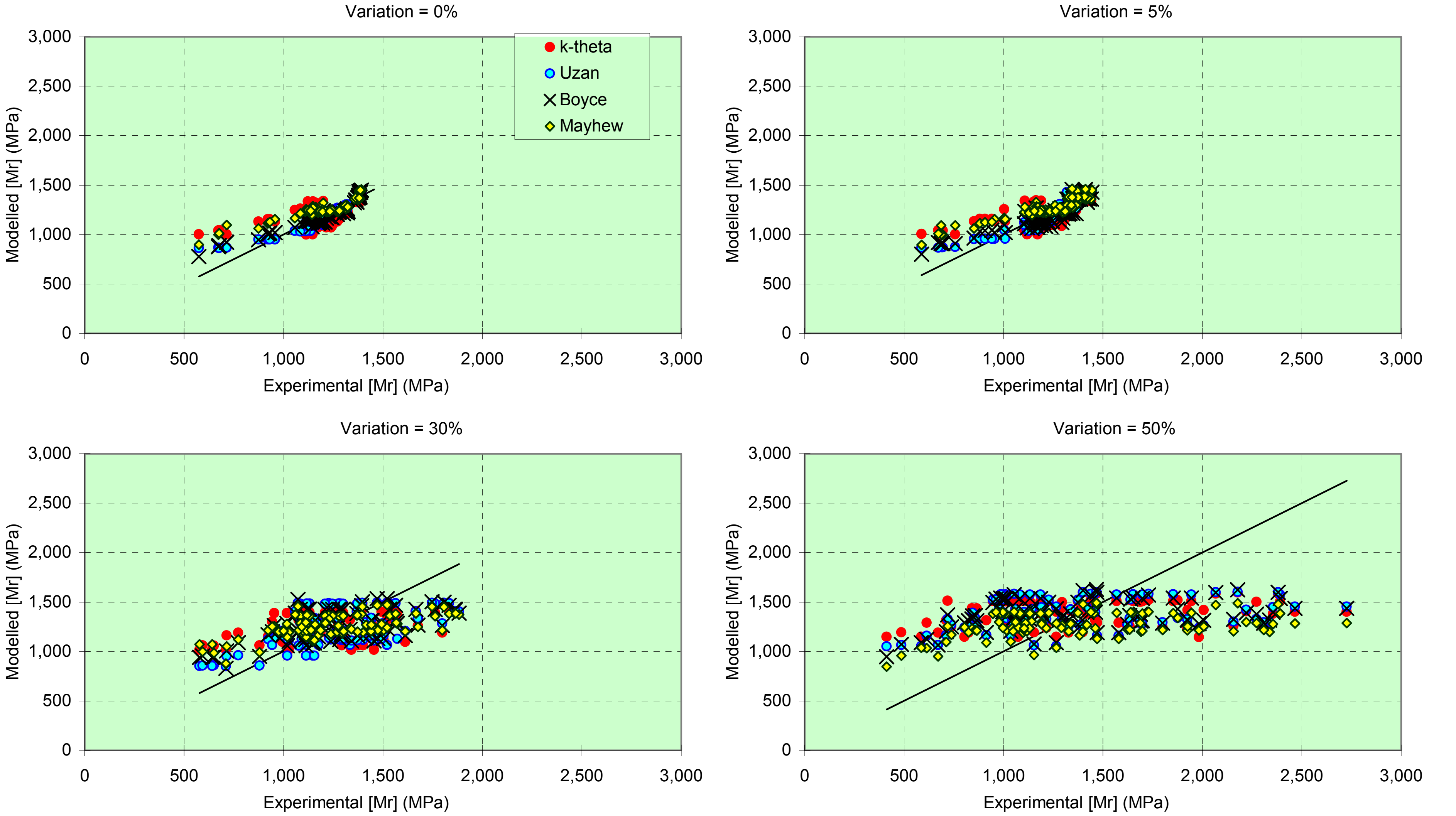


Figure 8-18 Increase in Scatter as the Variation Increases for an Unbound Granular Material



It was expected that the material coefficients would vary with the introduction of a random error of differing variation as shown in Figure 8-19 and Figure 8-20. There was, however, very little change in the coefficients, even when the variation was as great as 50%. It is thus concluded that the introduction of the random error will have little effect on characteristic resilient modulus. It is surmised that this is because random scatter does not affect the prediction of values by the models. This is important because it means that random errors that may be introduced into the results, for example by electronic noise in the data capture instruments, will not affect the final outcome of the analysis much. If this type of error was as large as 50% there would be a serious problem with the testing apparatus and in reality much smaller variations are expected. Therefore the models are not particularly sensitive to random variation in the results although it is noted that there is quite a large variation between the predicted resilient modulus between the actual models also shown in Figure 8-19 and Figure 8-20, therefore the difference in the final analysis could be considerable if one model was chosen over another. The solid line is based on a linear regression of the values over the range shown, 0% to 30%, whereas the dotted line is drawn to show the rapid increase over the range 30% to 50%.

Figure 8-19 Resilient Modulus with changing Error Variation for an Unbound Granular Material

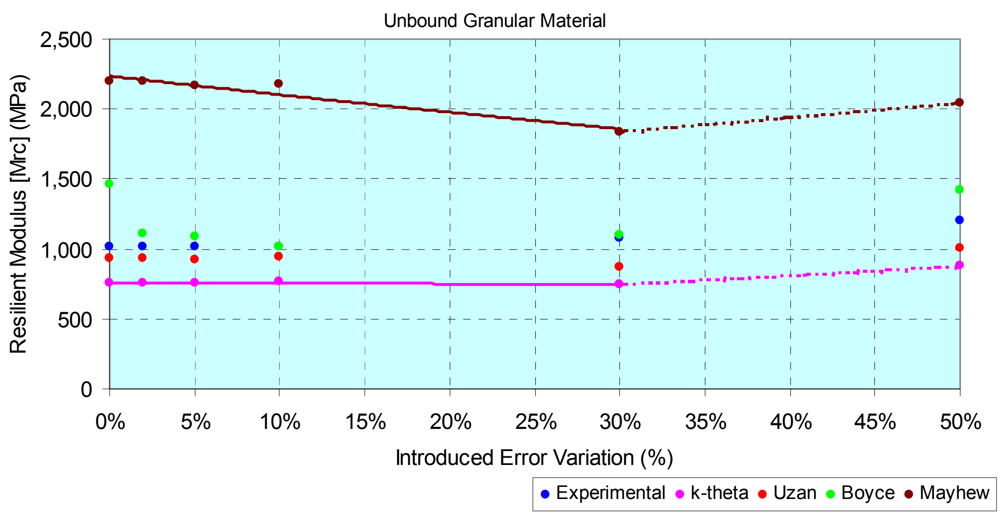
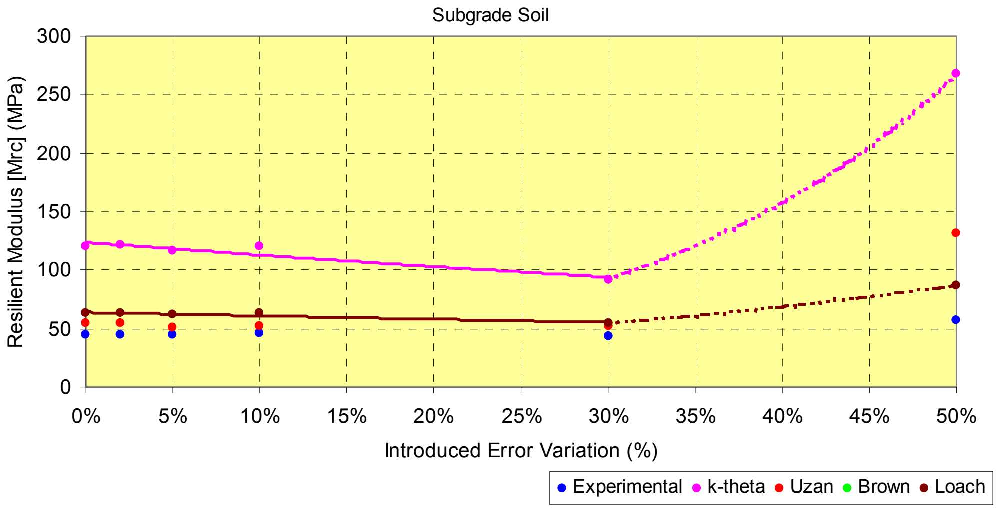


Figure 8-20 Resilient Modulus with changing Error Variation for a Subgrade Soil



8.5 Final Values from the Testing and Analysis

The final results from all three test programmes (material parameters and material coefficients) are used to analyse typical pavements in the next chapter.

The method of data verification and eliminating the outliers remains as discussed earlier. In all cases the material parameters are shown as an average of the verified data results and a 10th and 90th percentile value of the data. For Test Programme I, where a single specimen was tested at each laboratory, the results are quoted by material since they were calculated by taking the average values of all specimens and the deviation is taken as the 10th and 90th percentile values regardless of laboratory. For the other two test programmes, where more than one specimen was tested, for each material, at different laboratories, the results are quoted first by material then by laboratory and never combined. The final results of the analysis yielding the values and ranges of the parameters and coefficients, for the three test programmes, for all of the subgrade soils tested, are shown in Table 8-20 and for all of the unbound granular materials tested are shown in Table 8-21.

Table 8-20 Final Parameters and Coefficients for the Subgrade Soils

Test Pgm.	Mat.	Lab.	Experimental Values		Modelled Values												
					k-theta			Uzan				Brown			Loach		
			Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	A	B	Mr_c	C	D
Test Programme I	*Lower*	*10%*	*138*	*0.28*	*82*	*126,079*	*0.4037*	*99*	*226,959*	*0.2570*	*0.2145*	*147*	*84,026*	*-0.2594*	*147*	*2,843,389*	*0.7459*
	SFB	**All**	**160**	**0.41**	**96**	**143,380**	**0.3842**	**119**	**244,958**	**0.2226**	**0.1938**	**172**	**98,734**	**-0.2581**	**170**	**3,184,624**	**0.7333**
	Upper	*90%*	*190*	*0.55*	*117*	*170,183*	*0.3541*	*142*	*267,203*	*0.1799*	*0.1682*	*202*	*116,376*	*-0.2566*	*196*	*3,571,066*	*0.7191*
	Lower	*10%*	*23*	*0.53*	*33*	*28,851*	*-0.1559*	*20*	*10,465*	*0.1227*	*-0.6767*	*21*	*17,515*	*0.4508*	*21*	*43,787*	*1.4508*
	LOC	**All**	**34**	**0.54**	**45**	**33,040**	**-0.2594**	**30**	**14,036**	**-0.0284**	**-0.4879**	**27**	**21,858**	**0.4820**	**27**	**54,645**	**1.4820**
	Upper	*90%*	*44*	*0.56*	*61*	*38,750*	*-0.4004*	*42*	*18,458*	*-0.2155*	*-0.2540*	*31*	*25,304*	*0.5068*	*31*	*63,260*	*1.5068*
Test Programme II	*Lower*	*10%*	*6*	*0.51*	*8*	*4,429*	*-0.5620*	*7*	*2,409*	*-0.6398*	*0.1226*	*6*	*5,110*	*0.2186*	*6*	*12,774*	*1.2186*
	LOC	**LNEC**	**8**	**0.53**	**13**	**5,177**	**-0.7242**	**13**	**2,252**	**-0.7694**	**0.0458**	**8**	**6,587**	**0.3682**	**8**	**16,467**	**1.3682**
	Upper	*90%*	*12*	*0.58*	*21*	*6,461*	*-1.0026*	*22*	*1,988*	*-0.9882*	*-0.0836*	*12*	*8,870*	*0.5993*	*12*	*22,174*	*1.5993*
	Lower	*10%*	*15*	*0.41*	*19*	*23,542*	*0.1559*	*14*	*13,352*	*0.2010*	*-0.5256*	*13*	*9,871*	*0.8931*	*13*	*24,678*	*1.8931*
	LOC	**UNOT**	**22**	**0.68**	**43**	**21,273**	**-0.6034**	**23**	**12,518**	**-0.1223**	**-0.6058**	**21**	**17,512**	**0.5781**	**21**	**43,781**	**1.5781**
	Upper	*90%*	*29*	*0.96*	*66*	*19,132*	*-1.3199*	*34*	*11,584*	*-0.4841*	*-0.6955*	*30*	*25,480*	*0.2496*	*30*	*63,700*	*1.2496*
	Lower	*10%*	*45*	*0.63*	*99*	*40,556*	*-0.8336*	*51*	*11,351*	*-0.2865*	*-0.6545*	*33*	*24,152*	*0.7310*	*33*	*60,381*	*1.7310*
	LOC	**LRCF**	**48**	**0.74**	**112**	**43,376**	**-0.8888**	**58**	**11,597**	**-0.3231**	**-0.6660**	**36**	**25,647**	**0.7567**	**36**	**64,117**	**1.7567**
	Upper	*90%*	*51*	*0.84*	*123*	*45,699*	*-0.9343*	*63*	*11,785*	*-0.3510*	*-0.6748*	*38*	*26,922*	*0.7786*	*38*	*67,304*	*1.7786*
	Lower	*10%*	*20*	*0.37*	*30*	*17,492*	*-0.5121*	*17*	*6,637*	*0.1141*	*-0.8923*	*20*	*13,887*	*0.8442*	*20*	*34,718*	*1.8442*
	LOC	**DUT**	**23**	**0.42**	**37**	**19,736**	**-0.5906**	**21**	**6,866**	**0.0662**	**-0.9380**	**23**	**15,533**	**0.8972**	**23**	**38,832**	**1.8972**
	Upper	*90%*	*25*	*0.47*	*44*	*21,873*	*-0.6654*	*25*	*7,061*	*0.0255*	*-0.9767*	*25*	*16,738*	*0.9360*	*25*	*41,844*	*1.9360*
Test Programme III	*Lower*	*10%*	*93*	*0.22*	*91*	*54,817*	*-0.3212*	*121*	*60,030*	*-0.1499*	*-0.3988*	*100*	*86,325*	*0.3546*	*100*	*215,813*	*1.3546*
		25%	*105*	*0.26*	*149*	*73,617*	*-0.4512*	*165*	*61,451*	*-0.1815*	*-0.4510*	*135*	*106,367*	*0.4438*	*135*	*265,916*	*1.4438*
	LOC	**LNEC**	**161**	**0.33**	**223**	**97,303**	**-0.6150**	**263**	**64,625**	**-0.2520**	**-0.5675**	**204**	**146,365**	**0.6219**	**204**	**365,911**	**1.6219**
		75%	*200*	*0.37*	*264*	*110,590*	*-0.7069*	*344*	*67,249*	*-0.3103*	*-0.6638*	*247*	*171,613*	*0.7343*	*247*	*429,033*	*1.7343*
	Upper	*90%*	*229*	*0.50*	*385*	*149,561*	*-0.9764*	*453*	*70,800*	*-0.3891*	*-0.7942*	*386*	*252,161*	*1.0930*	*386*	*630,402*	*2.0930*
	Lower	*10%*	*141*	*0.22*	*111*	*126,210*	*0.0348*	*126*	*147,830*	*0.2086*	*-0.3514*	*150*	*106,795*	*0.4005*	*150*	*133,494*	*1.4005*
	LIM	**LNEC**	**226**	**0.32**	**275**	**187,006**	**-0.3015**	**328**	**135,299**	**-0.1238**	**-0.3973**	**247**	**135,632**	**0.5575**	**247**	**169,540**	**1.5575**
	Upper	*90%*	*337*	*0.45*	*406*	*235,484*	*-0.5696*	*669*	*114,132*	*-0.6853*	*-0.4749*	*340*	*163,230*	*0.7077*	*340*	*204,037*	*1.7077*

Table 8-21 Final Parameters and Coefficients for the Unbound Granular Materials

	Mat.	Lab.	Experimental Values		Modelled Values																		
					k-theta			Uzan				Boyce					Mayhew						
			Mr_e	υ_e	Mr_c	k_1	k_2	Mr_c	k_3	k_4	k_5	Mr_c	υ_c	Ga	Ka	n	Mr_c	υ_c	Ga	Ka	n	b	m
Test Programme I	Lower	10%	171	0.25	87	122,206	0.4363	117	227,011	0.2989	0.1446	90	-0.19	63,350	36,141	0.2428	111	0.27	62,075	59,651	0.2560	0.0411	0.4130
	SFB	All	200	0.35	103	140,508	0.4069	137	246,351	0.2635	0.1614	115	0.27	82,980	115,073	0.4006	121	0.47	78,350	217,163	0.3495	0.2813	0.7975
	Upper	90%	229	0.48	128	168,708	0.3617	164	273,733	0.2135	0.1853	140	0.78	112,838	233,881	0.5083	137	0.64	88,664	473,408	0.4547	0.5892	1.2997
	Lower	10%	452	0.33	236	364,985	0.5642	232	799,759	0.7274	-0.0900	180	-0.50	154,908	43,918	0.1123	207	0.59	160,537	21,295	0.0258	0.0409	0.1335
	CCD	All	554	0.40	311	416,892	0.4017	388	692,114	0.3126	0.1291	278	0.12	355,030	320,449	0.2978	269	0.62	172,857	97,841	0.2035	0.1969	0.6514
	Upper	90%	640	0.49	389	470,910	0.2326	525	597,979	-0.0502	0.3206	384	0.58	610,552	678,945	0.5216	332	0.66	185,178	174,388	0.3813	0.3529	1.1692
	Lower	10%	443	0.45	331	389,012	0.2805	420	505,501	0.2014	0.1199	397	-0.13	109,948	463,595	0.2437	359	0.27	104,399	972,395	-0.1005	0.0187	0.7252
	CCT	All	1,131	0.46	930	1,026,881	0.2035	1,323	1,229,127	0.0392	0.1886	1,318	0.11	234,974	2,123,377	0.3579	1,248	0.37	235,460	1,655,747	0.1811	0.2425	1.7719
	Upper	90%	1,820	0.47	1,529	1,664,750	0.1265	2,227	1,952,753	-0.1231	0.2573	2,240	0.35	360,001	3,783,159	0.4722	2,138	0.47	366,520	2,339,099	0.4627	0.4664	2.8185
	Lower	10%	Only One Specimen Tested																				
	MIG	All	235	0.28	79	133,321	0.6716	105	290,773	0.4896	0.1570	80	-0.24	114,963	11,175	0.0908	83	0.36	81,478	38,124	-0.0181	0.0595	0.1668
	Upper	90%	Only One Specimen Tested																				
Test Programme II	Lower	10%	178	0.14	80	150,359	0.8095	71	395,080	0.9426	-0.1032	86	-0.57	156,579	-912,637	0.1636							
	MIG	LNEC	218	0.22	99	174,492	0.7339	87	405,646	0.8502	-0.0896	103	-0.17	162,213	977,178	0.1161	122	0.38	NS	NS	NS	NS	NS
	Upper	90%	244	0.34	118	197,459	0.6620	106	417,387	0.7475	-0.0745	119	0.41	167,434	2,728,552	0.0721							
	Lower	10%	178	0.27	105	149,260	0.4325	74	254,106	0.7245	-0.2347	128	-0.81	-40,182	99,005	0.3493	125	0.41	123,576	70,757	0.2570	0.4307	2.0112
	MIG	UNOT	218	0.37	177	200,441	0.2430	148	262,752	0.4454	-0.1702	190	-0.23	622,015	69,324	0.6201	209	0.47	113,473	73,211	0.4367	0.4739	1.9558
	Upper	90%	261	0.46	259	259,432	0.0247	231	272,428	0.1329	-0.0980	261	0.33	1,371,897	35,713	0.9268	303	0.51	102,005	75,997	0.6405	0.5231	1.8929
	Lower	10%	215	0.29	138	184,723	0.3810	138	278,660	0.3964	-0.0197	108	-0.92	756,369	16,035	0.4494	159	0.01	145,388	79,052	0.3563	0.1322	0.4563
	MIG	LRSB	245	0.34	174	220,219	0.3140	181	306,985	0.2848	0.0358	177	-0.60	589,138	52,101	0.5499	212	0.25	187,846	89,619	0.4718	0.0966	0.5095
	Upper	90%	267	0.38	200	246,030	0.2652	213	327,489	0.2041	0.0759	255	-0.19	401,550	92,557	0.6627	257	0.43	224,174	98,661	0.5707	0.0660	0.5550
	Lower	10%	172	0.22	84	144,478	0.6793	84	293,553	0.6691	0.0105	88	-0.89	200,064	19,602	0.2755	197	0.20	495,334	94,027	0.2560	-1.6121	-1.2948
	MIG	DUT	198	0.23	116	173,821	0.5555	105	322,487	0.6268	-0.0618	123	-0.69	416,086	25,299	0.4091	203	0.44	285,341	94,187	0.3597	-0.8421	-0.5928
	Upper	90%	229	0.24	157	210,544	0.4006	125	349,171	0.5877	-0.1285	155	-0.45	623,184	30,761	0.5372	209	0.68	75,349	94,348	0.4633	-0.0721	0.1092
Test Prog III	Lower	10%	409	0.26	124	245,559	0.7461	28	318,306	1.0641	1.1616	205	-0.45	365,942	62,368	0.1864	178	0.15	3,385,321	201,533	0.1152	-0.2732	1.0650
		25%	491	0.32	158	278,668	0.7269	71	376,649	1.0390	1.1029	257	-0.21	373,822	79,087	0.2079	260	0.16	3,179,554	219,591	0.1585	-0.2604	1.0460
	CCD	LRSB	817	0.48	476	581,965	0.5508	381	803,897	0.8549	0.6730	682	-0.01	438,631	216,573	0.3848	552	0.25	2,439,090	284,574	0.3142	-0.2147	0.9777
		75%	992	0.64	525	629,116	0.5234	436	879,372	0.8223	0.5971	740	0.23	447,448	235,278	0.4088	615	0.27	2,278,385	298,677	0.3480	-0.2048	0.9628
	Upper	90%	1,348	0.69	1,043	1,123,288	0.2365	938	1,570,905	0.5244	-0.0987	1,453	0.38	556,113	465,804	0.7055	1,108	0.41	1,029,050	408,318	0.6108	-0.1276	0.8475
	Lower	10%	289	0.31	74	132,897	0.7465	65	389,970	0.9163	-0.1992	113	-0.89	325,886	15,486	0.1250	94	0.21	-64,620	66,103	-0.1634	-0.5983	0.4258
	CCT	LRSB	774	0.40	446	547,528	0.4772	426	753,290	0.5643	-0.0779	407	-0.79	673,201	69,736	0.3478	748	0.32	1,245,446	160,120	0.1411	-0.7012	0.5969
	Upper	90%	1,316	0.46	819	962,829	0.2074	857	1,187,048	0.1441	0.0670	770	-0.69	1,100,861	136,536	0.6221	1,517	0.44	2,788,027	270,823	0.4996	-0.8224	0.7984

8.6 SUMMARY

Constitutive relationships have been developed which attempt to model the behaviour of road construction material under traffic loading. The analysis of repeated load triaxial test data using these relationships yield material parameters and coefficients that describe the material characteristics under traffic loading.

Some simple material relationships are generally used in practice. More often, however, the material parameters (resilient modulus and Poisson's ratio) are obtained by unreliable relationships with empirical parameters or even just estimation with guidance from publications.

Having logged the data from the repeated load triaxial tests a method of data verification is proposed whereby all outliers are removed. Further, secondary screening of the data, outside defined percentile variation, is also conducted. Based on the removal of 'outliers' and the correlation of fit of the data the removal of the 10th percentile data appears to be a good compromise resulting in an improved correlation without the loss of too much data.

Analysis was conducted using a proprietary software package and a spreadsheet containing the method of least squares curve-fitting analysis. The spreadsheet provided a more robust method although it was limited in the statistical indicators provided.

Numerous analyses were conducted for a range of road construction materials resulting in the material properties (material parameters and model coefficients) being produced. Some problems in determining a realistic solution were encountered and a pragmatic set of rules (minimum and maximum values for the coefficients and parameters) was formulated (shown in Table 8-4). This allowed the characterisation of 'good' and 'bad' results, where the bad results were unrealistic and could be removed.

It order to determine material coefficients for all eventualities the coefficients were extrapolated for all values of characteristic resilient modulus for that particular sample, based on a linear regression of all points. This has allowed coefficients to be calculated for differing percentile values of the characteristic resilient modulus (i.e. 10%, 25% Average, 75% and 90%).

In general experimental values of resilient modulus for unbound granular materials were higher than the corresponding characteristic values obtained from the models, whereas for subgrade soils the opposite trend was observed. This implies that incorrect characteristic stresses may have been assumed and since the materials are stress dependent this could be corrected by increasing the characteristic stresses so that the characteristic resilient moduli also increased.

As expected, different laboratories, testing the same material, have resulted in different characteristic material properties. Also there is a difference in the results between specimens tested in the same laboratory at each of the four laboratories. The significance of these differences will be discussed in the next chapter.

9 DESIGN OF FLEXIBLE PAVEMENTS USING THE TEST RESULTS

9.1 INTRODUCTION

It was stated in an earlier chapter that the mechanistic design or analysis of flexible pavements refers to the numerical calculation of the deflection, stress and strain in a multi-layered pavement, when subjected to external loads, and the subsequent translation of these analytical calculations of pavement response into a prediction of the performance of the pavement.

During a parametric study Dawson and Plaistow (1993), analysing thinly surfaced flexible pavements using finite element analysis, suggested that a more detailed representation of granular material, in structurally significant unbound pavement layers would lead to more efficient (and thus more economical) pavement structures.

As has been discussed, it is well recognised the road construction materials are non-linear stress dependent anisotropic materials. Therefore it is best to characterise the material, by defining the material parameters in a model under realistic loading conditions. With the advent of powerful personal computers, the use of finite element analysis computer programs is one such technique.

Two computer analysis programs are used for this work. The first, ELSYM5 {Ahlborn (1963)}, is a commonly used simple non-linear multi-layer analysis program and the second is the more complex finite element analysis method used by FENLAP {Almeida (1991)} and developed at the University of Nottingham.

FENLAP is able to model the non-linearity of the layers, while in ELSYM5 (PC version) it is possible to simulate some non-linearity into a single layer by splitting the layer up into thinner layers and allocating different values of the material parameters at each layer. This program was, written some time ago, however, and it is unable to cope with more than five layers and this restricts it applicability somewhat.

The outputs are the same for each analytical program; namely strains and stresses at certain locations throughout the pavement structure. Typical European pavement structures have been chosen to undertake this analysis. These were defined during the 'Science Project'. The pavement structures and the points of analysis for the ELSYM computer analytical program are shown in Figure 9-1 and the analysis grid (as required for finite element analysis) for the FENLAP program is shown in Figure 9-2.

Figure 9-1 Analytical Points for ELSYM5

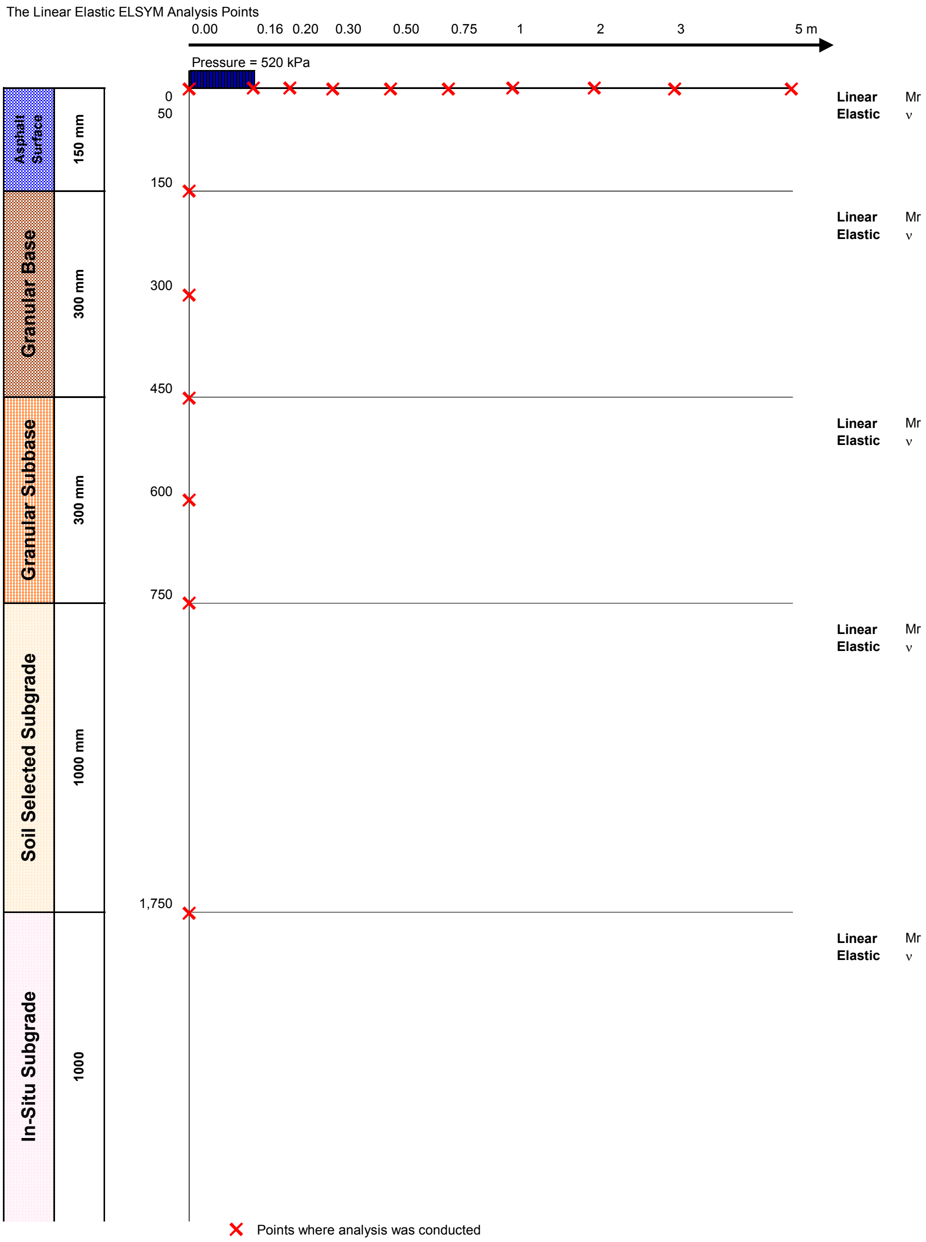
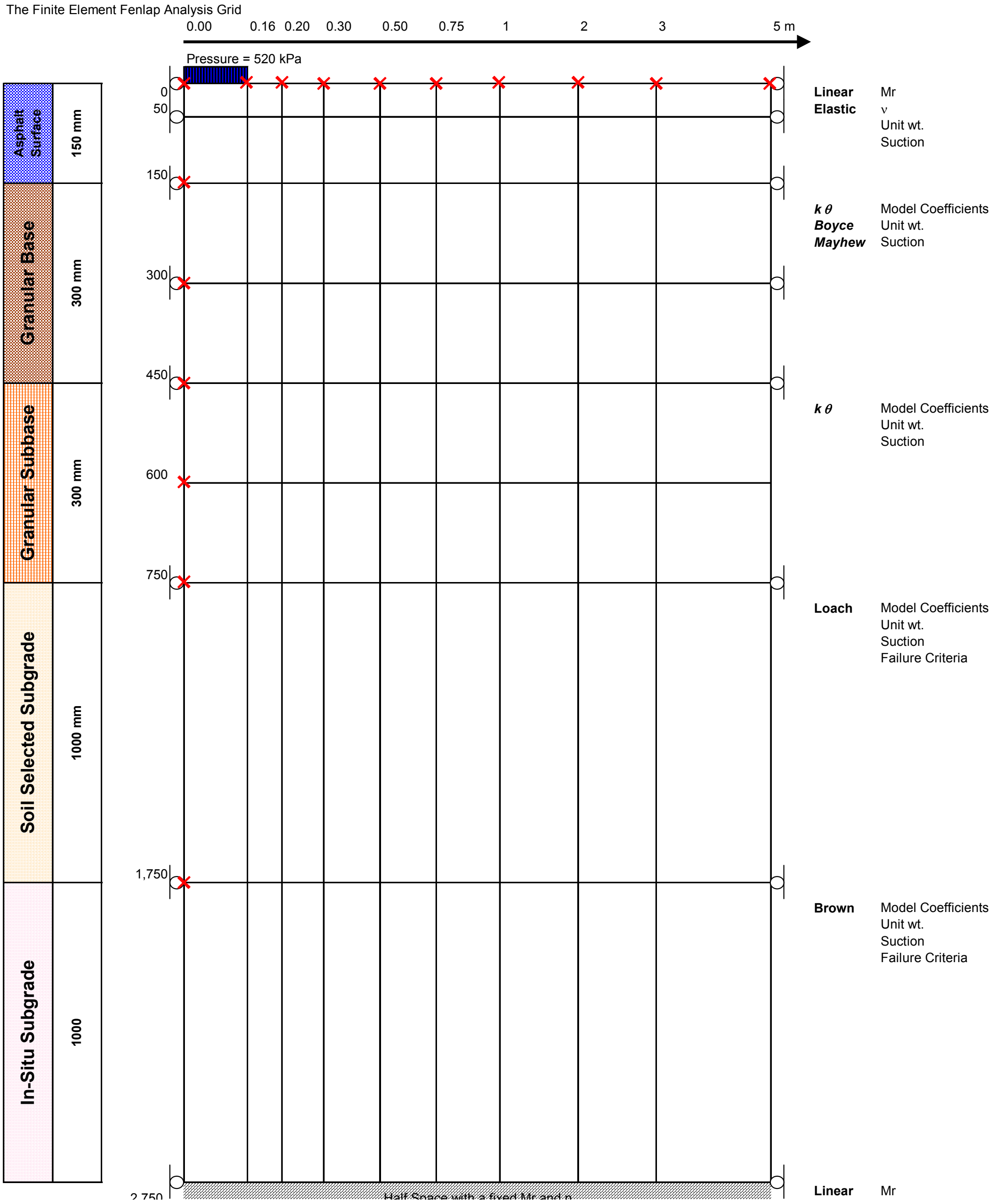


Figure 9-2 Analytical Grid for FENLAP



9.2 THE STRUCTURAL ANALYSIS OF SPECIFIC PAVEMENTS

The structural analysis of the pavement (determination of stresses and strains at particular points in the pavement under traffic loading) is performed by entering various data with respect to the pavement into an analytical computer program.

The required input is often, but not restricted to, the following:

- Number of layers and thickness (except for the lowest layer which is assumed to be semi-infinite);
- Number of loads and co-ordinates (x,y), tyre pressure and/ or contact area radius;
- Material parameters and/ or model coefficients;
- Co-ordinates of points to be investigated (x,y,z).

Depending on the nature of the material of each layer and its corresponding mechanism of behaviour, critical values of either stress or strain are selected as discussed earlier.

In the previous chapter, the material parameter values and the coefficients for the models that describe the material behaviour were determined. In order to compare the results, the materials, or more specifically the specimens from each laboratory and test programme, have been listed in order of characteristics resilient modulus as shown in Table 9-1. For this analysis it is necessary to identify a 'high quality' material from a 'poor quality' material and thus a 'strong' pavement structure, comprising high quality material from a 'weak' pavement structure comprising weaker material. This ranking is done by taking the higher characteristic resilient modulus to indicate better quality material. It is therefore expected that higher characteristic resilient moduli are found nearer the surface of the pavement structures, and also weaker pavements will have lower resilient moduli than stronger pavements.

During a comparison of these values against the guidelines in Chapter 2 it can be seen that SFB (granular material) and MIG have lower resilient moduli than expected. For the SFB this is likely since this material is not a base quality material but a fine-grained sand which has unbound granular properties. The reasons for the MIG showing these poor characteristics goes beyond the actual quality of the parent material, i.e. that as quarried and crushed and the explanation for the poor quality of this material may be that the unbound material has a poor particle size distribution or high clay content (PI). The other two granular materials (CCD and CCT) are shown to be medium to good quality base materials.

Table 9-1 Ranking of the Materials Tested in Terms of Quality

Test Programme	Material	Laboratory	Characteristic Resilient Modulus [Mr_c] (MPa)	
Unbound Granular Materials				
2	MIG	DUT	198	Poor Quality
1	SFB	All	200	
2	MIG	UNOT	218	
2	MIG	LNEC	218	
1	MIG	All	235	
2	MIG	LRSB	245	
1	CCD	All	554	
3	CCT	LRSB	774	
3	CCD	LRSB	817	
1	CCT	All	1131	Good Quality
Subgrade Soils				
2	LOC	LNEC	8	Poor Quality
2	LOC	UNOT	22	
2	LOC	DUT	23	
1	LOC	All	34	
2	LOC	LRCF	48	
1	SFB	All	160	
3	LOC	LNEC	161	
3	LIM	LNEC	226	Good Quality

The subgrade soils are unsuitable materials for base material in road construction. This is expected since this material is very fine grained and plastic. The Seine et Marne silt and the Fontainebleau sand are relatively good subgrade materials for road foundations.

The characteristic resilient modulus of these materials was found to vary between 8 and 226 MPa, probably dependent on moisture content, which is within the range for subgrade materials as discussed in Chapter 2. Thus, if these materials were to be incorporated in a pavement structure they would need to be kept as dry as possible.

In order to limit the number of analytical runs, it was necessary to limit the number of material relationships that would characterise each layer. It was decided to vary the models that describe the unbound granular base layer. The other layers were to have fixed material relationships. The pavement structure described in Table 9-2 was taken to be typical of those constructed in Europe.

Based on the values in Table 9-1 and Table 9-2 a comparison of the design lives for different basic pavement structures was conducted with the variations introduced as follows:

- Comparison 1 Variation of the base quality as defined from the test results of the base materials from the four different laboratories
- Comparison 2 Variation of the subgrade quality as defined from the test results of the base materials from the four different laboratories
- Comparison 3 Variation of base quality as defined from the test results from a single laboratory
- Comparison 4 Variation of the subgrade quality as defined from the test results from a single laboratory
- Comparison 5 Variation due to the introduction of random errors to the measuement data

Table 9-2 Pavement Structure and Characterisation Model for each Layer

Layer Description	Layer Thickness	Characterisation Model
Asphalt Surface	Varying thickness (50 – 150 mm)	• Linear elastic model
Unbound Granular Base	300 mm	• Linear elastic model • k-theta model • Boyce model • Mayhew model
Unbound Granular Subbase	300 mm	• Linear elastic model • k-theta model
Selected Soil Subgrade	1000 mm	• Linear elastic model • Loach model
Natural Soil Subgrade	Infinite depth	• Linear elastic model • Brown model

For each of these pavement structures the asphalt surface layer thickness was varied to take account of different pavements service states, as follows:

- Under Construction — 50 mm thick Asphalt
- In Service — 100 mm thick Asphalt; 150 mm thick Asphalt

Originally a surface thickness of 5 mm was included in order that a road without a surface could be simulated but FENLAP frequently returned an error with thinly surfaced pavement structures. It is thought that the reason for this is that the stresses become infinity high near the surface during the FENLAP analysis {Plaistow (1994)}. Therefore the 5 mm thinly surfaced pavement structure was dropped and the 50 mm surface was taken to be indicative of road under construction. FENLAP, however, frequently returned an error even for these pavement structures.

For the comparison of pavement structures when a random error was introduced an asphalt thickness of 100 mm was used.

An example of an analysis of the pavement structure is shown for the 'Under-Construction' case in Figure 9-3 and for the two 'In-Service' cases in Figure 9-4 and

Figure 9-5. It can be seen in these figures that the only difference between each of these pavement structures is the thickness of the asphalt surface layer.

For each different pavement configuration and circumstances there were a total of five different analytical runs with varying models and load characteristics as follows:

- Linear elastic model analysis for all layers with dual wheel loads of 2 x 20 kN each — ELSYM-D
- Linear elastic model analysis for all layers with a single wheel load of 40 kN — ELSYM-S
- Stress dependent model throughout with k-theta model describing the base with a single wheel load of 40 kN — Fenlap-K
- Stress dependent model throughout with Boyce model describing the base with a single wheel load of 40 kN — Fenlap-B
- Stress dependent model throughout with Mayhew model describing the base with a single wheel load of 40 kN — Fenlap-M

The method used to determine the pavement life from the strain predictions at specific locations in the pavement structure is that of the SA-MDM as described in an earlier chapter. The results are, therefore, presented in terms of Equivalent Standard Axles (ESA) also defined in an earlier chapter and this can be seen on the figures on the following pages. The critical traffic loading in ESA is shown in red and reproduced at the bottom of the figure. It can be seen that for the three examples shown the failure occurs in the surfacing for all three cases, however with the greater thickness of surface the predicted traffic loading increases as follows:

Asphalt Thickness (mm)	**Equivalent Standard Axles (ESA)**
50	950
100	13,500
150	141,600

Figure 9-3 Mechanistic Analysis of the Pavement Structures Under Construction (50 mm Asphalt Surface)

Pavement Structure - Under Construction (UC):

Surface	**50 mm**	Bituminous prime coat to protect the granular base						**Unit Weight: 23 kN/m³**
		Mr_i= 2,100 MPa **ν_i= 0.40**						**Suction: 0 kPa**
		Peak Deflection on the Surface		ElsymD	ElsymS	FenlapK	FenlapB	FenlapM
			δ_p (mm)=	0.587	0.741	2.855	NS	2.914
		Strain - Bottom of the Base Layer		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 49 mm	ElsymD	266	308	-463		1.582E+06
			ElsymS	273	273	-464		2.673E+06
			FenlapK	1560	1560	-2160		1.286E+03
			FenlapB	NS	NS	NS		
	50		FenlapM	1670	1670	-2290		**9.535E+02**
DUT - MIG	**300 mm**	Unbound Granular Base	Test programme: 2			Lab.: DUT		MIG
		Char.: **Mr_i= 198 MPa** **ν_i= 0.23**						**Unit Weight: 22 kN/m³**
		k-theta: **k_1= 1,738** **k_2= 0.5555**						**Suction: 0 kPa**
		Boyce: **Ka= 1,664** **Ga= 27,374**	**n= 0.4091**					
		Mayhew: **Ka= 4,936** **Ga= 14,953**	**n= 0.3597**			**m= -0.5928**		**β= -0.8421**
		Stress - Centre of the Base Layer		σ_1(kPa)	σ_2(kPa)	σ_3(kPa)	FOS	ESA
		Depth= 200 mm	ElsymD	133	1	-11	14.6	1.085E+46
			ElsymS	201	-11	-11	11.4	4.635E+36
			FenlapK	166	-14	-15	11.6	1.302E+37
			FenlapB	NS	NS	NS	NS	
	350		FenlapM	84	-21	-23	12.7	2.825E+40
All - MIG	**300 mm**	Unbound Granular Subbase	Test programme: 1			Lab.: All		MIG
		Char.: **Mr_i= 235 MPa** **ν_i= 0.28**						**Unit Weight: 22 kN/m³**
		k-theta: **k_1= 1,333** **k_2= 0.6716**						**Suction: 0 kPa**
		Stress - Centre of the Base Layer		σ_1(kPa)	σ_2(kPa)	σ_3(kPa)	FOS	ESA
		Depth= 500 mm	ElsymD	41	-4	-7	19.2	1.676E+59
			ElsymS	52	-8	-8	15.3	1.016E+48
			FenlapK	83	6	6	16.0	8.088E+49
			FenlapB	NS	NS	NS	NS	
	650		FenlapM	62	6	6	19.6	3.595E+60
All - SFB	**1000 mm**	Selected Soil Subgrade	Test programme: 1			Lab.: All		SFB
		Char.: **Mr_i= 160 MPa** **ν_i= 0.41**						**Unit Weight: 16 kN/m³**
		Loach: **C= 31,846** **D= 0.7333**	**q_f =**	**1.71**	***p +**	**26**		**Suction: 3 kPa**
		Strain - Top of the Selected Subgrade		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 651 mm	ElsymD	-53	-71	131		4.457E+14
			ElsymS	-80	-80	164		3.348E+13
			FenlapK	-7	-7	35		1.858E+21
			FenlapB	NS	NS	NS	NS	
	1650		FenlapM	-4	-4	28		2.005E+22
All - LOC	**Infinite**	In-Situ Soil Subgrade Foundation	Test programme: 1			Lab.: All		LOC
		Char.: **Mr_i= 34 MPa** **ν_i= 0.54**	**=0.45**					**Unit Weight: 22 kN/m³**
		Brown: **A= 21,858** **B= 0.4820**	**q_f =**	**0.001**	***p +**	**86**		**Suction: 32 kPa**
		Strain - Top of the Subgrade Layer		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 1651 mm	ElsymD	-37	-39	61		2.790E+18
			ElsymS	-39	-39	62		2.209E+18
			FenlapK	-12	-12	29		1.345E+22
			FenlapB	NS	NS	NS	NS	
	2650		FenlapM	-12	-12	28		2.362E+22
						Minimum Design Loading:		**9.535E+02**

NS – No Solution

Figure 9-4 Mechanistic Analysis of the Pavement Structures In Service (100 mm Asphalt Surface)

Pavement Structure - In-Service (IS):

Layer	Thickness	Description						
Surface	**100 mm**	Asphalt Surface and Base Layer						**Unit Weight:** **23 kN/m³**
		Mr_i= 2,100 MPa **ν_i= 0.40**						**Suction: 0 kPa**
		Peak Deflection on the Surface		ElsymD	ElsymS	FenlapK	FenlapB	FenlapM
			δ_p (mm)=	0.461	0.567	1.778	1.127	1.482
		Strain - Bottom of the Base Layer		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 99 mm	ElsymD	179	256	-332		3.586E+06
			ElsymS	290	290	-444		2.073E+06
			FenlapK	913	913	-1230		**1.347E+04**
			FenlapB	732	732	-965		3.551E+04
	100		FenlapM	883	883	-1180		1.560E+04
DUT - MIG	**300 mm**	Unbound Granular Base	Test programme: 2			Lab.: DUT		MIG
		Char.: **Mr_i= 198 MPa**	**ν_i= 0.23**					**Unit Weight:** **22 kN/m³**
		k-theta: **k_1= 1,738**	**k_2= 0.5555**					**Suction: 0 kPa**
		Boyce: **Ka= 1,664**	**Ga= 27,374**	**n= 0.4091**				
		Mayhew: **Ka= 4,936**	**Ga= 14,953**	**n= 0.3597**		**m= -0.5928**		**β= -0.8421**
		Stress - Centre of the Base Layer		*σ_1 (kPa)*	*σ_2 (kPa)*	*σ_3 (kPa)*	FOS	ESA
		Depth= 250 mm	ElsymD	86	-1	-8	17.4	1.302E+54
			ElsymS	123	-9	-9	13.3	1.508E+42
			FenlapK	69	1	1	20.4	7.148E+62
			FenlapB	47	-16	-17	16.9	4.438E+52
	400		FenlapM	45	-9	-9	20.3	2.994E+62
All - MIG	**300 mm**	Unbound Granular Subbase	Test programme: 1			Lab.: All		MIG
		Char.: **Mr_i= 235 MPa**	**ν_i= 0.28**					**Unit Weight:** **22 kN/m³**
		k-theta: **k_1= 1,333**	**k_2= 0.6716**					**Suction: 0 kPa**
		Stress - Centre of the Base Layer		σ_1(kPa)	σ_2(kPa)	σ_3(kPa)	FOS	ESA
		Depth= 550 mm	ElsymD	32	-5	-7	21.6	2.461E+66
			ElsymS	39	-7	-7	18.1	1.009E+56
			FenlapK	44	6	6	25.4	1.883E+77
			FenlapB	31	7	7	36.2	9.526E+108
	700		FenlapM	32	5	5	32.7	4.432E+98
All - SFB	**1000 mm**	Selected Soil Subgrade	Test programme: 1			Lab.: All		SFB
		Char.: **Mr_i= 160 MPa**	**ν_i= 0.41**					**Unit Weight:** **16 kN/m³**
		Loach: **C= 31,846**	**D= 0.7333**	**q_f =**	**1.71**	***p +**	**26**	**Suction: 3 kPa**
		Strain - Top of the Selected Subgrade		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 701 mm	ElsymD	-47	-60	110		3.405E+15
			ElsymS	-66	-66	132		4.015E+14
			FenlapK	-1	-1	19		1.552E+24
			FenlapB	1	1	14		7.269E+25
	1700		FenlapM	1	1	15		2.589E+25
All - LOC	**Infinite**	In-Situ Soil Subgrade Foundation	Test programme: 1			Lab.: All		LOC
		Char.: **Mr_i= 34 MPa**	**ν_i= 0.54**	**=0.45**				**Unit Weight:** **22 kN/m³**
		Brown: **A= 21,858**	**B= 0.4820**	**q_f =**	**0.001**	***p +**	**86**	**Suction: 32 kPa**
		Strain - Top of the Subgrade Layer		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 1701 mm	ElsymD	-34	-36	56		8.136E+18
			ElsymS	-36	-36	57		5.718E+18
			FenlapK	-11	-11	26		5.815E+22
			FenlapB	-10	-10	24		1.337E+23
	2700		FenlapM	-10	-10	24		1.275E+23
						Minimum Design Loading:		**1.347E+04**

NS – No Solution

Figure 9-5 Mechanistic Analysis of the Pavement Structures In Service (150 mm Asphalt Surface)

Pavement Structure - In-Service (IS):

Layer	Thickness / Depth	Description						
Surface	**150 mm**	Asphalt Surface and Base Layer						**Unit Weight: 23 kN/m³**
		Mr_i= 2,100 MPa **ν_i= 0.40**						**Suction: 0 kPa**
		Peak Deflection on the Surface		ElsymD	ElsymS	FenlapK	FenlapB	FenlapM
			δ_p (mm)=	0.394	0.465	1.232	0.811	0.924
		Strain - Bottom of the Base Layer		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 149 mm	ElsymD	121	185	-229		1.498E+07
			ElsymS	222	222	-332		6.706E+06
			FenlapK	534	534	-707		**1.416E+05**
			FenlapB	383	383	-522		6.084E+05
	0		FenlapM	479	479	-633		2.281E+05
DUT - MIG	**300 mm**	Unbound Granular Base	Test programme: 2		Lab.: DUT			MIG
		Char.: **Mr_i= 198 MPa**	**ν_i= 0.23**					**Unit Weight: 22 kN/m³**
		k-theta: **k_1= 1,738**	**k_2= 0.5555**					**Suction: 0 kPa**
		Boyce: **Ka= 1,664**	**Ga= 27,374**		**n= 0.4091**			
		Mayhew: **Ka= 4,936**	**Ga= 14,953**		**n= 0.3597**		**m= -0.5928**	**β= -0.8421**
		Stress - Centre of the Base Layer		σ_1(kPa)	σ_2(kPa)	σ_3(kPa)	FOS	ESA
		Depth= 300 mm	ElsymD	61	-2	-6	20.4	8.174E+62
			ElsymS	82	-7	-7	15.9	6.199E+49
			FenlapK	36	2	2	33.3	3.691E+100
			FenlapB	49	-17	-17	16.0	9.286E+49
	450		FenlapM	28	-6	-6	28.0	1.333E+85
All - MIG	**300 mm**	Unbound Granular Subbase	Test programme: 1		Lab.: All			MIG
		Char.: **Mr_i= 235 MPa**	**ν_i= 0.28**					**Unit Weight: 22 kN/m³**
		k-theta: **k_1= 1,333**	**k_2= 0.6716**					**Suction: 0 kPa**
		Stress - Centre of the Base Layer		σ_1(kPa)	σ_2(kPa)	σ_3(kPa)	FOS	ESA
		Depth= 600 mm	ElsymD	25	-5	-6	25.0	2.108E+76
			ElsymS	29	-6	-6	21.6	1.760E+66
			FenlapK	26	5	5	39.4	1.534E+118
			FenlapB	28	5	5	36.0	1.902E+108
	750		FenlapM	21	4	4	46.8	6.104E+139
All - SFB	**1000 mm**	Selected Soil Subgrade	Test programme: 1		Lab.: All			SFB
		Char.: **Mr_i= 160 MPa**	**ν_i= 0.41**					**Unit Weight: 16 kN/m³**
		Loach: **C= 31,846**	**D= 0.7333**		**q_f = 1.71**	***p + 26**		**Suction: 3 kPa**
		Strain - Top of the Selected Subgrade		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 751 mm	ElsymD	-41	-50	91		2.804E+16
			ElsymS	-54	-54	107		4.678E+15
			FenlapK	3	3	11		1.498E+27
			FenlapB	1	1	12		3.610E+26
	1750		FenlapM	3	3	10		4.293E+27
All - LOC	**Infinite**	In-Situ Soil Subgrade Foundation	Test programme: 1		Lab.: All			LOC
		Char.: **Mr_i= 34 MPa**	**ν_i= 0.54**	**=0.45**				**Unit Weight: 22 kN/m³**
		Brown: **A= 21,858**	**B= 0.4820**		**q_f = 0.001**	***p + 86**		**Suction: 32 kPa**
		Strain - Top of the Subgrade Layer		$\varepsilon_x(\mu\varepsilon)$	$\varepsilon_y(\mu\varepsilon)$	$\varepsilon_z(\mu\varepsilon)$		ESA
		Depth= 1751 mm	ElsymD	-31	-33	72		4.429E+17
			ElsymS	-33	-33	75		2.749E+17
			FenlapK	-10	-10	23		2.672E+23
			FenlapB	-10	-10	22		3.457E+23
	2750		FenlapM	-9	-9	21		5.582E+23
						Minimum Design Loading:		**1.416E+05**

NS – No Solution

It may be argued that 950 ESA is unrealistically low but this is a possible failing of the design method and outside the scope of this work. It does show that the permissible traffic loading increases with asphalt surface thickness as would be expected.

The mechanistic analyses details for all of the work in this chapter are contained in Appendix H.

9.3 THE INFLUENCE OF THE MATERIAL VARIATIONS TO PAVEMENT DESIGN

The mechanistic pavement analyses are summarised in Table 9-3, where the different material combinations for each pavement design are shown. The red text indicates the layers where the variations were made.

The first and second analyses varied the quality of the base material (Microgranite) and the subgrade (London Clay) based on the different results obtained from the four different during Test Programme II. The variation was from the results that predicted the poorest quality (DUT for Microgranite and LNEC for London Clay) to the best quality (LRSB for Microgranite and LRCF for London Clay) shown in Table 9-3. The characteristic resilient modulus values used to rank the materials are shown in Table 9-1.

The second comparison was to take the results of a series of test conducted at a single laboratory and to investigate the effect of the variation of the results obtained. Again a pavement structure was selected and first the base material (Hard Limestone) was varied followed by the subbase material (London Clay). The laboratory tests for the base material were conducted at LRSB and those of the subgrade at LNEC as part of Test Programme III. For each of these two materials the range of the test results obtained at the particular laboratory was divided into 10%ile, 25%'ile, average, 75%'ile and 90%'ile values as shown in Table 9-3. The actual resilient modulus values resulting from the laboratory tests were shown in the previous chapter. Considering that each analysis had three different asphalt thicknesses and five different analytical modelling methods 270 analyse runs were conducted. However, not all analyses were successful, it was found that FENLAP did not always find a solution and this is shown in Table 9-4.

Table 9-3 Pavement Structures with Different Material Characteristics that were Analysed

Structure	Material	Selection	Pavement Life as a function of the Variation in the Base Characteristics				Pavement Life as a function of the Variation in the Subgrade Characteristics				Pavement Life as a function of the Range of values in the Base Characteristics for a Single Result					Pavement Life as a function of the Range of values in the Subgrade Characteristics for a Single Result				
Analysis Run:	Pavement Quality		Poor → Good				Poor → Good				Poor → Good					Poor → Good				
	Pavement No.		5	6	7	8	9	10	11	12	10%	25%	Avg.	75%	90%	10%	25%	Avg.	75%	90%
	Quality within range		Avg.	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.										
Surface	Asphalt		Fixed Mr and ν				Fixed Mr and ν				Fixed Mr and ν					Fixed Mr and ν				
Base	Unbound Granular Material	Material	**MIG**	**MIG**	**MIG**	**MIG**	CCT				**CCD**					CCD				
		Laboratory	**DUT**	**LNEC**	**UNOT**	**LRSB**	LRSB				**LRSB**					LRSB				
		T.Prog.	**TP2**	**TP2**	**TP2**	**TP2**	TP3				**TP3**					TP3				
Subbase	Unbound Granular Material	Material	MIG				MIG				MIG					MIG				
		Laboratory	All				DUT				DUT					DUT				
		T.Prog.	TP1				TP2				TP2					TP2				
SSG	Subgrade Soil	Material	SFB				**LOC**	**LOC**	**LOC**	**LOC**	LOC					**LOC**				
		Laboratory	ALL				**LNEC**	**UNOT**	**DUT**	**LRCF**	LNEC					**LNEC**				
		T.Prog.	TP1				**TP2**	**TP2**	**TP2**	**TP2**	TP3					**TP3**				
SG	Subgrade Soil	Material	LOC				LOC				LOC					LOC				
		Laboratory	ALL				ALL				UNOT					UNOT				
		T.Prog.	TP1				TP1				TP2					TP2				

Red text shows the layer that the coefficients were varied

Table 9-4 Analyses Conducted showing when Successful Solutions were Achieved

Asphalt Thickness (mm)	Models	Pavement Life as a function of the Variation in the Base Characteristics				Pavement Life as a function of the Variation in the Subgrade Characteristics				Pavement Life as a function of the Range of values in the Base Characteristics for a Single Result					Pavement Life as a function of the Range of values in the Subgrade Characteristics for a Single Result				
Analysis Run:	Pav'nt Q'ty	Poor →			Good	Poor →			Good	Poor →				Good	Poor →				Good
	Pav'nt No.	5	6	7	8	9	10	11	12	10%	25%	Avg.	75%	90%	10%	25%	Avg.	75%	90%
	Quality	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.										
50	ELYSM D	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	ELSYM S	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP K	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP B	✗	✗	✗	✗	✓	✗	✗	✗	✓	✓	✗	✗	✗	✓	✓	✗	✗	✗
	FENLAP M	✓	✗	✓	✗	✓	✗	✗	✓	✓	✓	✗	✗	✗	✗	✗	✗	✗	✗
100	ELYSM D	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	ELSYM S	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP K	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP B	✓	✗	✗	✗	✓	✓	✓	✓	✓	✓	✓	✓	✗	✓	✓	✓	✗	✓
	FENLAP M	✓	✗	✓	✗	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
150	ELYSM D	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	ELSYM S	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP K	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP B	✓	✗	✗	✗	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	FENLAP M	✓	✗	✓	✗	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Unsuccessful Runs	ELYSM D																		
	ELSYM S																		
	FENLAP K																		
	FENLAP B	1	3	3	3		1	1	1			1	1	2			1	2	1
	FENLAP M		3		3		1	1				1	1	1	1	1	1	1	1
	Total	1	6	3	6		2	2	1			2	2	3	1	1	2	3	2

The results of this table are summarised in Table 9-5, of the 270 analytical runs a total of 37 (14%) were unsuccessful.

Table 9-5 Summary of the Mechanistic Analysis Run Results

Analytical Model Name	Model Type	Unsuccessful Runs	
		No.	%
ELSYM – D (Dual Load)	Linear Elastic	0	0%
ELSYM – S (Single Load)	Linear Elastic	0	0%
FENLAP – K	k-theta – model	0	0%
FENLAP – B	Boyce - model	21	8%
FENLAP – M	Mayhew - model	16	6%

This shows that the more simple linear analysis method is much more likely to provide an answer to the problem, of course the accuracy of these answers may be questionable. When FENLAP is used to analyse the pavements absolute success is found for the more simple k-theta model; the Boyce model yields the worst degree of success and the Mayhew slightly better than the Boyce model. Of the 37 failures 25 were found on the Under-Construction case (50 mm asphalt surface) while 12 were for the In-Service case (100 and 150 mm asphalt surface).

A further 40 mechanistic analyses were conducted to investigate the influence of the introduction of a random error of differing variance as shown in Table 9-6. Since the material parameters and coefficients were found to vary little (chapter 8) not all of the analyses were conducted since it was felt that little difference would result. Analytical runs were conducted for a variance of 0%; 5%; 30% and 50%. As discussed in the previous chapter two materials were selected for this analysis a granular material – Hard Limestone and a subgrade soil – London Clay as shown in Table 9-6.

Table 9-6 Mechanistic Analysis with Varying Material Characteristics showing Successful Solutions

Structure	Material	Selection	Pavement life as a function of the Range of values in the Base Characteristics when a Random Error is Introduced into the Strain Measurements	Pavement life as a function of the Range of values in the Subbase Characteristics when a Randon Error is Introduced into the Strain Measurements
Analysis Run:		Variation Quality	Little → Large	Little → Large
		Variation	0% 2% 5% 10% 30% 50%	0% 2% 5% 10% 30% 50%
Surface	Asphalt		Fixed Mr and ν	Fixed Mr and ν
Base	Unbound Granular Material	Material	**CCD**	CCD
		Laboratory	**LRSB**	LRSB
		T.Prog.	**TP3**	TP3
Subbase	Unbound Granular Material	Material	MIG	MIG
		Laboratory	DUT	DUT
		T.Prog.	TP2	TP2
SSG	Subgrade Soil	Material	LOC	**LOC**
		Laboratory	LNEC	**LNEC**
		T.Prog.	TP3	**TP3**
SG	Subgrade Soil	Material	LOC	LOC
		Laboratory	UNOT	UNOT
		T.Prog.	TP2	TP2

Red text shows the layer that was varied

Of the 40 analytical runs 7 (18%) were unsuccessful, all of those using the Boyce model, as shown in Table 9-7.

Table 9-7 Mechanistic Analysis with Random Errors Introduced showing Successful Solutions

Asphalt Thickness (mm)	Models	Pavement life as a function of the Range of values in the Base Characteristics when a Random Error is Introduced into the Strain Measurements						Pavement life as a function of the Range of values in the Subbase Characteristics when a Randon Error is Introduced into the Strain Measurements					
Analysis Run:	Var. Q'ty	Little Variance			Large Variance			Little Variance			Large Variance		
	Variation	0%	2%	5%	10%	30%	50%	0%	2%	5%	10%	30%	50%
100	ELYSM D	✓	Not Analysed	✓	Not Analysed	✓	✓	✓	Not Analysed	✓	Not Analysed	✓	✓
	ELSYM S	✓		✓		✓	✓	✓		✓		✓	✓
	FENLAP K	✓		✓		✓	✓	✓		✓		✓	✓
	FENLAP B	✗		✗		✗	✗	✗		✗		✓	✗
	FENLAP M	✓		✓		✓	✓	✓		✓		✓	✓
Unsuccesful Runs	ELYSM D												
	ELSYM S												
	FENLAP K												
	FENLAP B	1		1		1	1	1		1			1
	FENLAP M												
	Total	1		1		1	1	1		1			1

9.3.1 Comparison 1 - Variation of the Base Strength from Four Different Laboratories

During Test Programme II, an unbound granular base material (Microgranite) was tested at four different laboratories. Three test specimens were fabricated to strict properties, moisture content and density, at each laboratory. Stresses and strains were measured during the repeated load triaxial test in accordance with a detailed test procedure and analysed as described. From the analysis material parameters and coefficients were obtained for the specimens from each laboratory.

A pavement structure was chosen with fixed properties for the surface, subbase and subgrade layers and the quality of the base varied by the results from each laboratory. The detailed mechanistic analyses results are contained in Appendix H and tables containing the predicted traffic life in ESA are shown this appendix. These results are summarised in Figure 9-6.

As stated in Chapter 2 most common pavement design methods are primarily concerned with the two critical strain values namely the horizontal tensile strain at the bottom of the asphalt layer (to limit asphalt fatigue cracking) and the vertical compressive strain at the top of the subgrade (to prevent excessive permanent deformation). The SA-MDM method, which is used here, also considers shear deformation and failure in the unbound granular layers and since this method was selected for use here, all three criteria are considered. Of course, the lesser ESA allowed according to each of these three chosen criteria determines the limiting pavement life. For the pavements chosen here it was found that, almost exclusively, the pavement life is determined on the basis of asphalt tensile strain, indicating that fatigue at the bottom of the asphalt layer is the critical failure criterion. However, it is noted that other failure methods, such as permanent strain in aggregate layers or in the subgrade, which are not critical for these pavement structures may be critical for other pavement structures and design methods. Thus the conclusions drawn from this study might not be universally applicable.

Figure 9-6 Comparison 1 - Variation of the Base Strength from Four Different Laboratories in Test Programme III

Pavement Life as a function of the Variation in the Base Characteristics

Life (ESA X10[6]) 50 mm Asphalt @ 2100 MPa

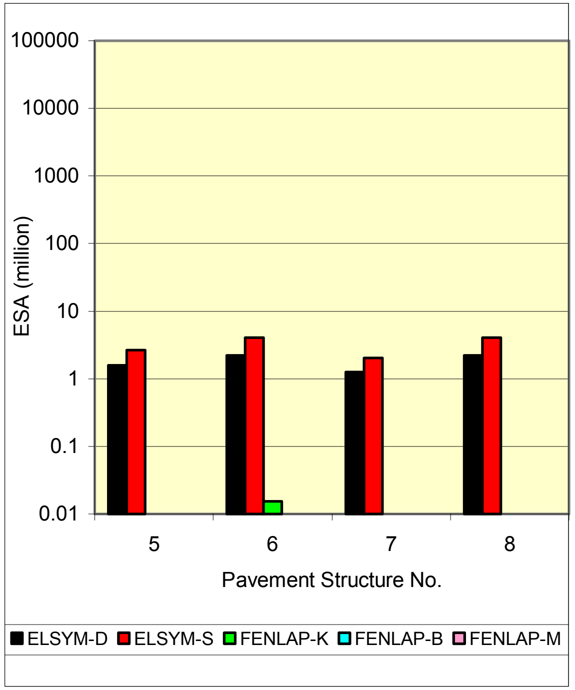
	Pavement No.	5	6	7	8
	Laboratory	DUT	LNEC	UNOT	LRSB
Model	ELSYM-D	1.58	2.22	1.26	2.21
	ELSYM-S	2.67	4.09	2.04	4.06
	FENLAP-K	0.00	0.02	0.00	0.00
	FENLAP-B				
	FENLAP-M	0.00		0.00	

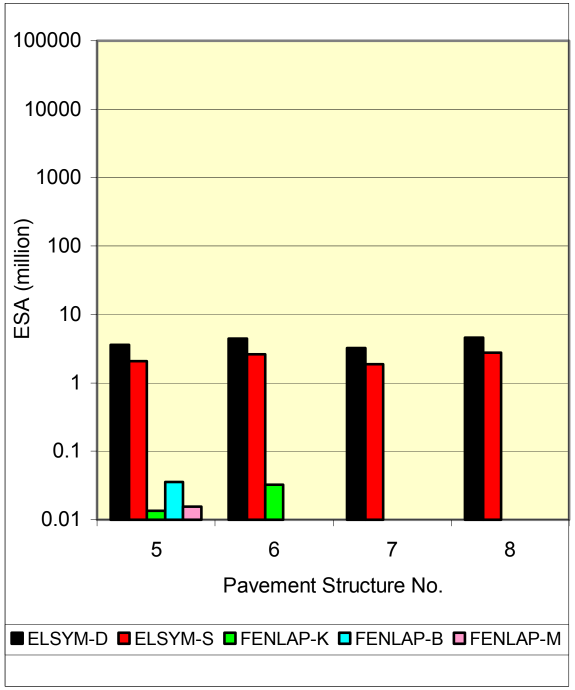
Life (ESA X10[6]) 100 mm Asphalt @ 2100 MPa

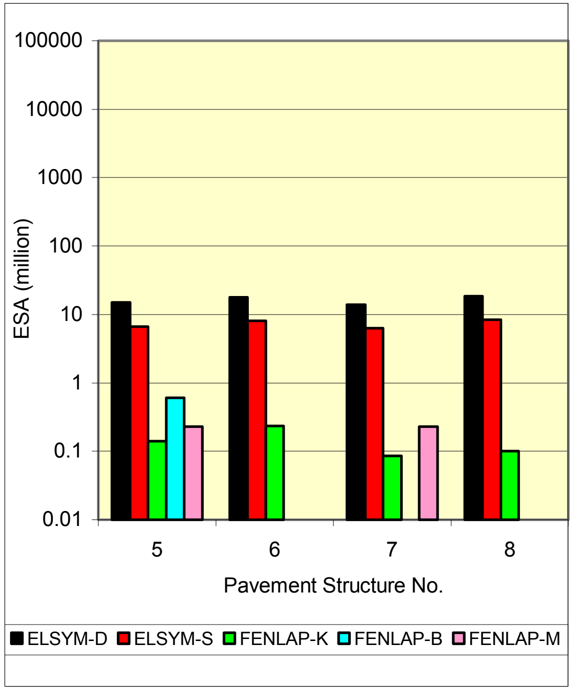
	Pavement No.	5	6	7	8
	Laboratory	DUT	LNEC	UNOT	LRSB
Model	ELSYM-D	3.59	4.45	3.25	4.61
	ELSYM-S	2.07	2.63	1.88	2.76
	FENLAP-K	0.01	0.03	0.01	0.01
	FENLAP-B	0.04			
	FENLAP-M	0.02		0.00	

Life (ESA X10[6]) 150 mm Asphalt @ 2100 MPa

	Pavement No.	5	6	7	8
	Laboratory	DUT	LNEC	UNOT	LRSB
Model	ELSYM-D	14.98	17.84	13.99	18.51
	ELSYM-S	6.71	8.01	6.26	8.33
	FENLAP-K	0.14	0.23	0.09	0.10
	FENLAP-B	0.61			
	FENLAP-M	0.23		0.23	







Pavement Structure No. corresponds to those pavements listed in the earlier tables

Clearly, the pavement life increases with increasing thickness in the asphalt surface layer. Taking the FENLAP-K model, for example, the average traffic prediction for the pavement with 50 mm asphalt is 4,200 ESA, 100 mm is 14,700 ESA and 150 mm is 140,000 ESA all of which are fairly low in terms of pavement life. These are vastly different, however, from the prediction made by the ELSYM-D model, which predicts the average traffic prediction for the pavement with 50 mm asphalt is 1.8×10^6 ESA, 100 mm is 4.0×10^6 ESA and 150 mm is 16.3×10^6 ESA, which appear to be much more realistic estimations.

It is noted that the base is of poor quality (resilient modulus between 198 and 245 MPa) by the standards set in Chapter 2 and this may account for the low predicted lives. Also, the variation between laboratories is quite small as discussed in the previous chapter. It is also noted that the third pavement structure (Nos.11; 12; 13 - UNOT) predicts a lower life than the second structure (Nos.8; 9; 10 - LNEC). These pavements have the same characteristic resilient modulus but the predicted Poisson's ratio for the third pavement is greater than that for the second (0.37 against 0.22). This emphasises the importance of estimating this material parameter accurately when it is required as an input, i.e. linear elastic and k-theta models in this work.

In summary there is little change in the predicted life from a single pavement structure with increasing base quality, which implies one of two things, namely:

- The differences in the test results from each of the four laboratories test results is insignificant with respect to this mechanistic pavement design, or;
- That the modelling process does not provide realistic predictions of material behaviour which is supported by the vast difference in predictions between the models, this may be due to the complex nature of these materials in pavement structures.

Results were not obtained for all of the model types, since many Boyce and Mayhew models failed to produce a result, for this pavement structure combination.

It is noted that the equivalent dual and single wheel loads (ELSYM) produce varying results. In general under the single wheel load the life of the pavement is extended for thinly surfaced pavements. This is thought to be due to a concentration of stress under the dual load at the bottom of the thin surface (which is critical), which would occur in the centre or upper part (mainly comprehensive zone) of the thicker surfaced pavements.

For variation in the base quality and the use of different models, the following points are noted:

- The linear elastic analysis (ELSYM) using the dual and single tyre loads of the same stress provide similar results;
- The Boyce and Mayhew models are largely unsuccessful when analysing this pavement structure, however the k-theta model is always successful;
- FENLAP predicts almost immediate failure, particularly for the thinly surfaced roads, ELSYM predicts almost 100 times the life that FENLAP predicts, clearly one is incorrect;
- There does not appear to be a trend between the quality of the base in terms of characteristic resilient modulus and the predicted traffic loading (although there is only a small change in characteristic resilient modulus).

9.3.2 Comparison 2 - Variation of the Subgrade Strength from Four Different Laboratories

Similarly the comparison was undertaken by applying the variation to a subgrade soil (London Clay) tested at four different laboratories. Again, three test specimens were fabricated at each laboratory to the same strict properties. Stresses and strains were measured and material parameters and coefficients obtained for the specimens from each laboratory. A pavement structure was chosen whereby the surface, base and subbase were given common parameters but this time the subgrade was varied with the results from each laboratory. The actual mechanistic analyses are contained in Appendix H and tables containing the predicted traffic life in ESA are shown in this appendix. These results are summarised in Figure 9-7.

Figure 9-7 Comparison 2 - Variation of the Subgrade Strength from Four Different Laboratories

Pavement Life as a function of the Variation in the Subgrade Characteristics

Life (ESA X10^6) 50 mm Asphalt @ 2100 MPa

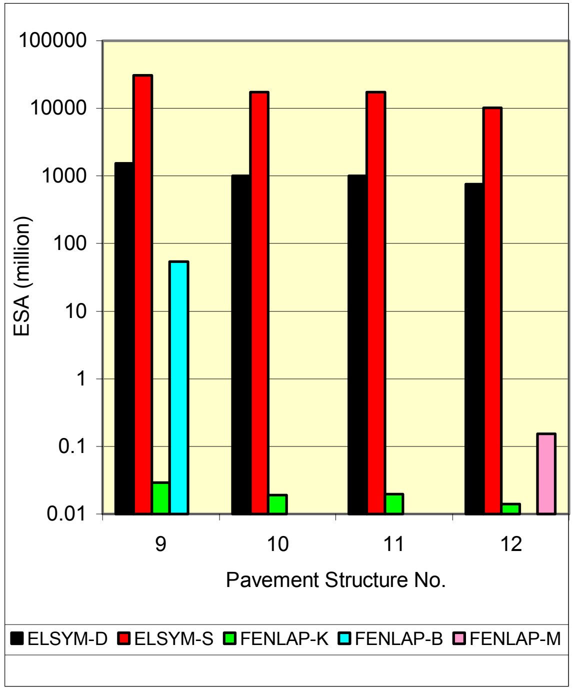
	Pavement No.	9	10	11	12
	Laboratory	LNEC	UNOT	DUT	LRCF
Model	ELSYM-D	1533	1009	1013	755
	ELSYM-S	30779	17361	17340	10287
	FENLAP-K	0.03	0.02	0.02	0.01
	FENLAP-B	53.53			
	FENLAP-M	0.00			0.15

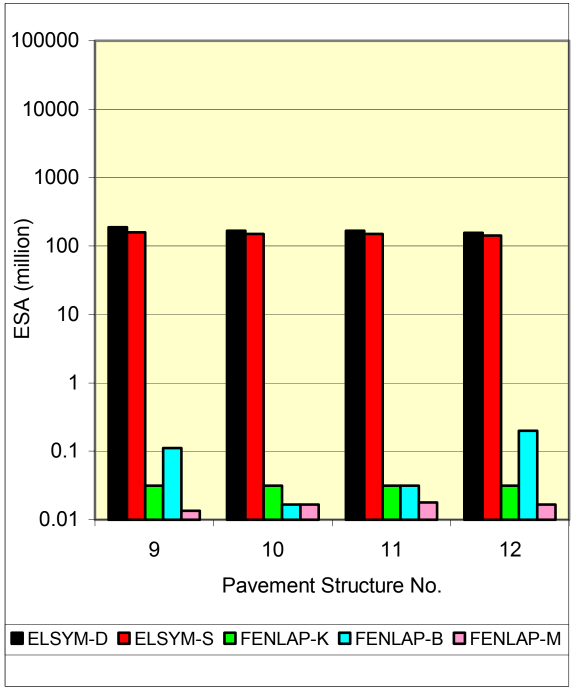
Life (ESA X10^6) 100 mm Asphalt @ 2100 MPa

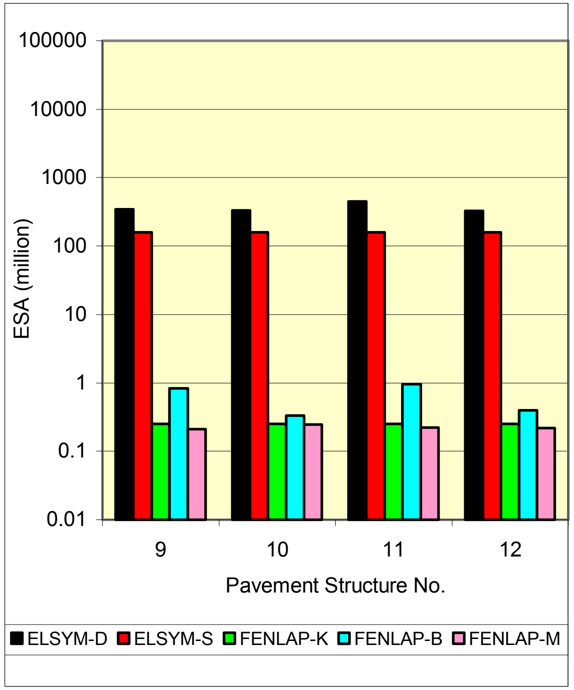
	Pavement No.	9	10	11	12
	Laboratory	LNEC	UNOT	DUT	LRCF
Model	ELSYM-D	187	167	168	156
	ELSYM-S	159	151	151	142
	FENLAP-K	0.03	0.03	0.03	0.03
	FENLAP-B	0.11	0.02	0.03	0.20
	FENLAP-M	0.01	0.02	0.02	0.02

Life (ESA X10^6) 150 mm Asphalt @ 2100 MPa

	Pavement No.	9	10	11	12
	Laboratory	LNEC	UNOT	DUT	LRCF
Model	ELSYM-D	343	330	445	326
	ELSYM-S	159	158	159	158
	FENLAP-K	0.25	0.25	0.25	0.25
	FENLAP-B	0.84	0.33	0.95	0.40
	FENLAP-M	0.21	0.25	0.22	0.22







Pavement Structure No. corresponds to those pavements listed in the earlier tables

Again there was little change in the predicted life from one pavement structure to another, although the subgrade material quality is changed quite significantly, by a factor of 6 (8 to 48 MPa). This may be due to the fact that the influence of the subgrade layer, being further down in the pavement structure, is less significant than that for materials that are closer to the surface.

The non-linear FENLAP analyses show an increase of pavement life with increasing thickness of the asphalt surface layer (with the exception of FENLAP-B - 50 mm asphalt thickness case) and this is expected. However, the ELSYM linear elastic analyses show a consistent decrease in predicted life from 50 mm to 100 mm and a small increase from 100 mm to 150 mm and this is not what one would expect. Obviously this model is not accurately depicting the real situation since not only is it expected that longer lives are obtained from thicker asphalt surfaces but also more realistic results are obtained for thicker asphalt surfaces due to better understanding of these materials and their behaviour under loading. Thin asphalt layers are more flexible and, although early cracking may occur, these layers are more able to cope with higher deflections and thus the life of the pavement structure is greater than if thicker surfaces were used. This is because stiff layers 'attract' higher stresses, however they distribute them better than thin layers. ELSYM considers each layer as a continuum in bending and is therefore concerned only with the stress build-up at the bottom of the layer, however, for thin asphalt layers, shear may be more significant than tensile strain. Since changes in thickness, when asphalt thickness is small, may cause large changes in attracted stress as well as large changes in flexibility it is expected that tensile fibre strain, on which fatigue life is based, would change rapidly. Therefore, for larger surface thicknesses the flexibility of the layer will drop as the thickness is increase without much change in attracting stress so extreme fibre strain will drop and consequently the life will increase.

The FENLAP-B, with 50 mm surfacing, life is anomalous in all aspects (See Figure 9-6 and Figure 9-7) and a computational/ numerical problem seems likely given the non-convergent solutions of pavements Number 8, 11 and 14 (Figure 9-7).

With respect to the two thicker surfaced pavements, for which all of the analyses were successful, there is little difference in the predicted lives between the particular pavement structures with varying subgrade quality. However, there is an enormous difference between the predictions made by the ELSYM and FENLAP analysis methods. The average ELSYM life prediction for a pavement with a 100 mm thick asphalt layer is 160 x 10^6 ESA as opposed to the 46,000 ESA for the FENLAP and similarly for the 150 mm thick asphalt surface the ELSYM life prediction is 260 x 10^6 ESA as opposed to the FENLAP prediction of 370,000 ESA.

A check was conducted on these pavements by plotting the surface deflection bowls for all models for the two cases (pavement 15 – 100 mm and 16 – 150 mm) as shown in Figure 9-8 and Figure 9-9. For the 100 mm surface the maximum non-linear surface deflection was approximately 3 times that for the linearly elastic simulation, and similarly, for the 150 mm surface pavement, a factor of at least 2 is found. Deflection bowls have been plotted for all pavement analyses and can be found in Appendix H. It is thought that the FENLAP analysis does not realise the full potential of the materials and the predictions are low. Unfortunately, a true deflection bowl was not measured on a pavement constructed with the relevant structure. Had this been conducted it would be possible to verify the analytical predictions.

Figure 9-8 Surface Deflection Bowls for a Pavement Structure with a 100 mm Asphalt Surface

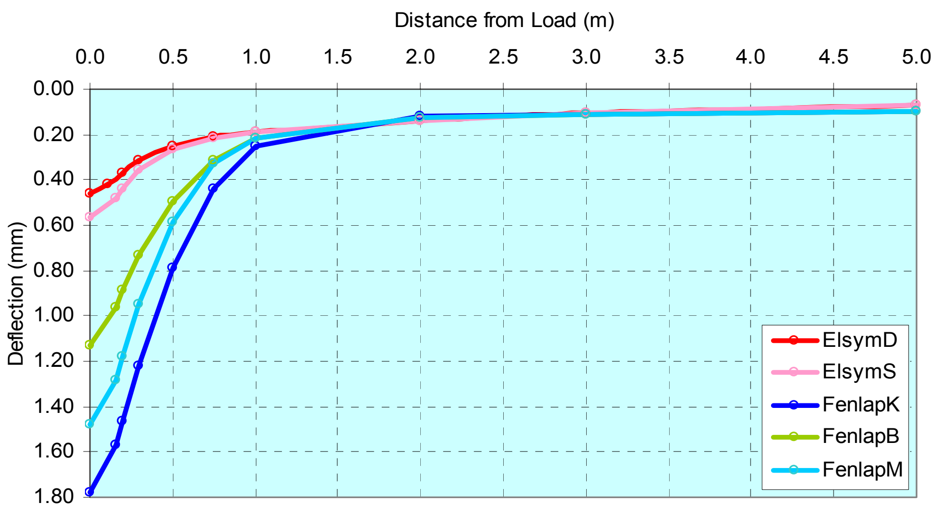
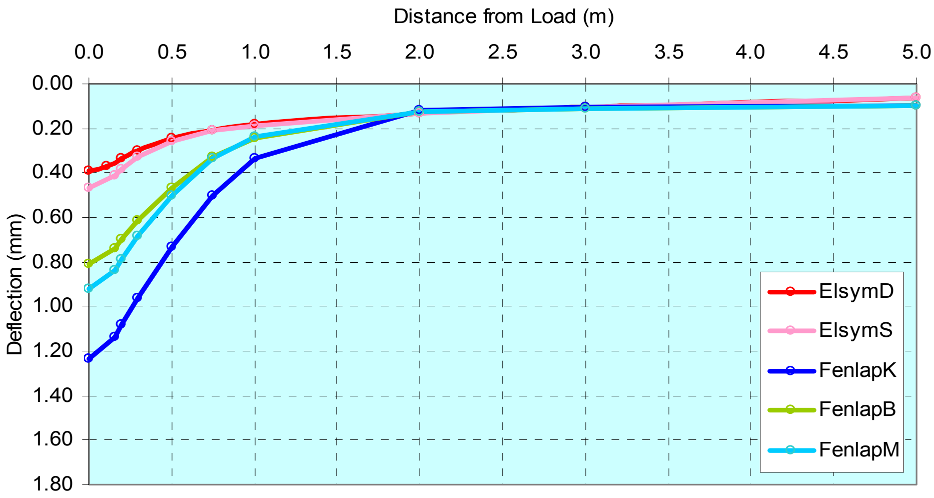


Figure 9-9 Surface Deflection Bowls for a Pavement Structure with a 150 mm Asphalt Surface



The above differences are unacceptable and as such it is probable that both of the two analysis methods do not make correct predictions. As with the earlier analyses ELSYM predicts a much greater life, almost 1,000 times greater than the life that FENLAP predicts, and one is, at least, incorrect. The SA-MDM was used to compile the South African structural pavement design guideline {Committee of State Road Authorities (CSRA) (1983)} and owing to lower expected loading over the design life, the pavement structures in this guideline are considerably thinner that those used for this work. Therefore, it is difficult to make a definite comparison between the guideline structure and those analyses in this work. A pavement, however, shown in this guideline with a 40 mm thick asphalt surface, a 150 mm granular base and 150 mm granular subbase is predicted to withstand between 0.8 and 3.0 millions ESA. This range assumes that the subgrade foundation has a soaked bearing capacity of CBR>15% which according to the dubious relationship discussed earlier (Equation 2-1) is of the correct order of magnitude when compared to pavement structures that were used in the variation of the base characteristics in this chapter (i.e. Pavement No. 5 to 8). The pavements structures used for the analyses herein have two times the thickness of granular base and subbase and from 50 mm, 100 mm and 150 mm, i.e. up to three times, the surface thickness, therefore it seems unlikely that the low predictions made by FENLAP (< 0.1million ESA) are realistic. The ELSYM analysis predicts pavement lives of the same order for the pavements analysed (1.5 to 4.0 million ESA) which is also low considering the thicker base and subbase. Further, ELSYM predicts little improvement in the design life with increase in asphalt surface thickness which also seems incorrect.

It is well established that road construction materials are non-linear inelastic in nature under loading and therefore the linear elastic methods used in ELSYM are obviously incorrect since this method treats each layer as a beam and as such allows some tension (or effective tension) to be present in the analysis. Another possibility is that FENLAP is basically modelling the situation correctly but that the high strain predicted is not, in reality, the cause of failure. For example, if the layer is pulling apart, not bending, it would have a high tensile strain without deflecting in the same manner as that predicted by ELSYM. Under this scenario the top of the layer might be in tension too and ELSYM would not predict this. Further, this method does not allow any change in the material characteristics under loading horizontally and, since it is a system of definite layers, it assumes a perfect bond between layer interfaces which may not be correct. An advantage of the linear elastic method is that it has been in use for a long time, ELSYM since 1963, and is still widely used today. It is consequently the basis for many established pavement design methods (including the SA-MDM used here). Therefore, although the modelling method may not be entirely accurate, the prediction of traffic loading based on these methods has been validated and subsequently modified over the past years.

The more sophisticated finite element methods and mathematical models used by FENLAP undoubtedly model the stresses and strains in the material layers better than the ELSYM approach which, for example, often predicts tension at the bottom of unbound layers. The finite element approaches, however, lack the wide and long term use which results in validation against real pavement performance. These approaches also introduce some artifices in their modelling method, for example, in order to prevent the occurrence of tension at the bottom of granular materials the following techniques are used:

- It is possible to reduce the stiffness as the compressive stress approaches zero which will result in very large lateral strains;
- Allow large plastic deformation to occur at the bottom of the layer which will allow the material to stretch laterally and thus no tension will occur;
- The horizontal stress is increased dramatically at the bottom of the layer in order to cancel out apparent tension (the technique employed by FENLAP).

This may make the materials and pavements less likely to fail under repeated loading than is found in practice. Thus neither FENLAP nor ELSYM are likely to provide an accurate replication of reality. Their different approaches may account for the huge difference between results from these different methods.

Based on these analyses, using the two analysis methods chosen ELSYM and FENLAP , the following observations are made:

- The linear elastic analysis (ELSYM) using the dual and single tyre loads of the same magnitude of stress provide similar results.
- The Boyce and Mayhew models are largely unsuccessful when analysing the thinly surfaced (50 mm) pavement structure. The k-theta model, however, is always successful.
- ELSYM predicts a much greater design lives than FENLAP for these roads. The ELSYM predictions appear to be more realistic.
- There does not appear to be a clear relationship between the quality of the base, in terms of characteristic resilient modulus, and the predicted traffic loading.

9.3.3 Comparison 3 and 4- Variation of the range of Values of the Base and Subgrade Material Characteristics Conducted at a Single Laboratory

During the analysis and presentation of the material parameters and coefficients in the previous chapter, it was stated that the 10th and 90th percentile values were calculated for all values based on the characteristic resilient modulus. For a set of results (CCD and LOC in Test Programme III) the 25th and 75th percentile results were also calculated together with the average value (as used in the analyses above). These values have been applied to two mechanistic analyses, one with varying base properties and the other varying the subgrade properties. The detailed mechanistic analyses are contained in Appendix H and tables containing the predicted traffic life in ESA are shown in this appendix. These results are summarised in Figure 9-10 and Figure 9-11.

Although the same conclusions with respect to the difference between models apply, for both analyses, it can clearly be seen that the variation in the base layer has a large effect on the predicted life of the pavement whereas the variation in the subgrade has little or no effect. This substantiates the hypothesis that the importance of accurately determining the properties of the upper layers is greater that that of the lower layers.

Observation made for these comparisons are:

- The linear elastic analysis (ELSYM) using the dual and single tyre loads of the same stress provide similar results.
- Again ELSYM normally predicts a much greater life for these roads, often 1,000 times the life that FENLAP predicts, and suggestions for this difference have been given.
- The variation in the base is much more critical than that of the subgrade, as discussed.

Figure 9-10 Comparison 3 - Variation within the Range of Values for the Base Strength at a Single Laboratory

Pavement Life as a function of the Range of values in the Base Characteristics for a Single Result

Life (ESA X10^6) 50 mm Asphalt @ 2100 MPa

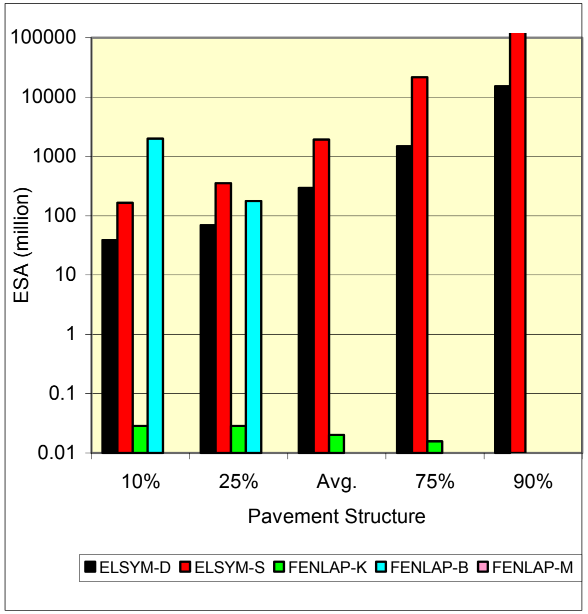
	Variation	10%	25%	Avg.	75%	90%
Model	ELSYM-D	38.6	69.1	292	1478	15317
	ELSYM-S	165	350	1912	21696	1556093
	FENLAP-K	0.03	0.03	0.02	0.02	0.00
	FENLAP-B	1978	177.5			
	FENLAP-M	0.01	0.01			

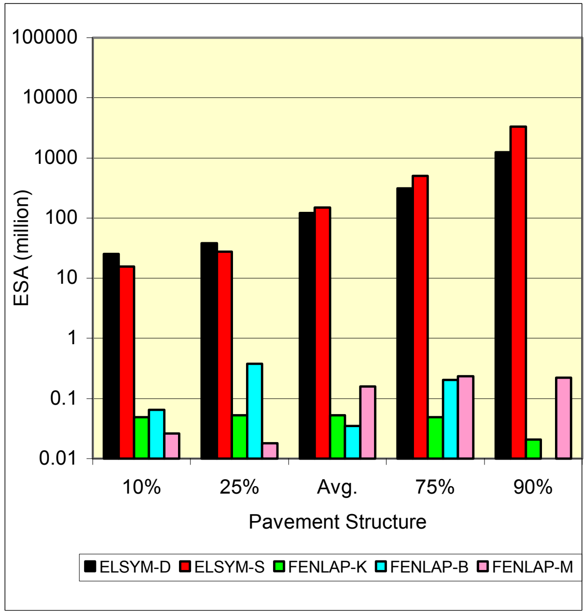
Life (ESA X10^6) 100 mm Asphalt @ 2100 MPa

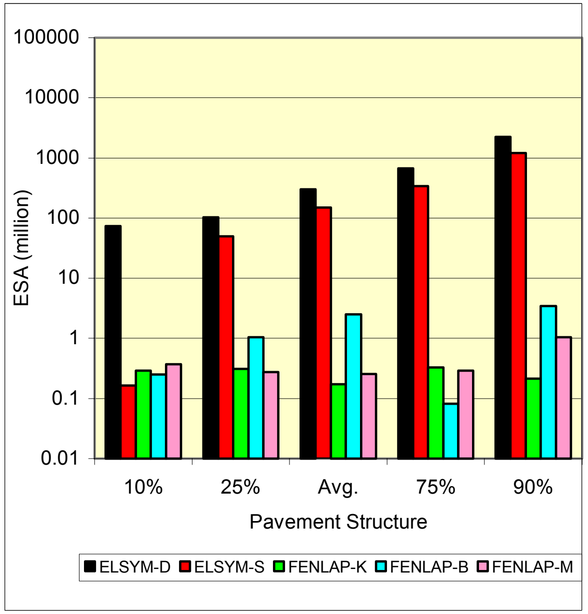
	Variation	10%	25%	Avg.	75%	90%
Model	ELSYM-D	25.4	38.1	122.3	313.3	1251
	ELSYM-S	15.6	27.6	150.6	504.7	3339
	FENLAP-K	0.05	0.05	0.05	0.05	0.02
	FENLAP-B	0.06	0.38	0.03	0.20	
	FENLAP-M	0.03	0.02	0.16	0.23	0.22

Life (ESA X10^6) 150 mm Asphalt @ 2100 MPa

	Variation	10%	25%	Avg.	75%	90%
Model	ELSYM-D	73.7	104	302	669	2251
	ELSYM-S	0.2	49.6	149	341	1200
	FENLAP-K	0.29	0.31	0.17	0.33	0.21
	FENLAP-B	0.25	1.04	2.52	0.08	3.44
	FENLAP-M	0.37	0.27	0.25	0.29	1.05







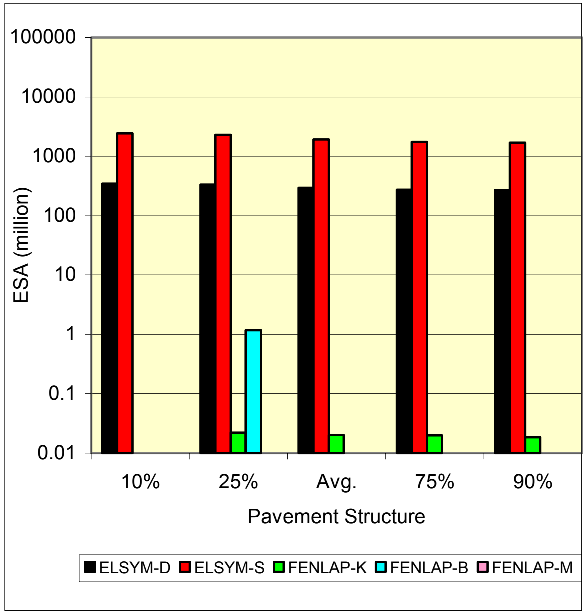
Pavement Structure No. corresponds to those pavements listed in the earlier tables

Figure 9-11 Comparison 4 - Variation within the Range of Values for the Subgrade Strength at a Single Laboratory

Pavement Life as a function of the Range of values in the Subgrade Characteristics for a Single Result

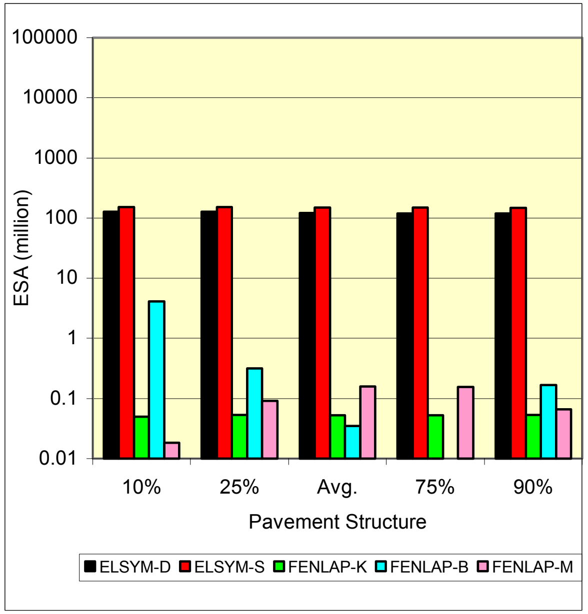
Life (ESA X10^{6}) 50 mm Asphalt @ 2100 MPa

	Variation	10%	25%	Avg.	75%	90%
Model	ELSYM-D	343	332	292	275	267
	ELSYM-S	2432	2312	1912	1752	1695
	FENLAP-K	0.00	0.02	0.02	0.02	0.02
	FENLAP-B		1.17			
	FENLAP-M					



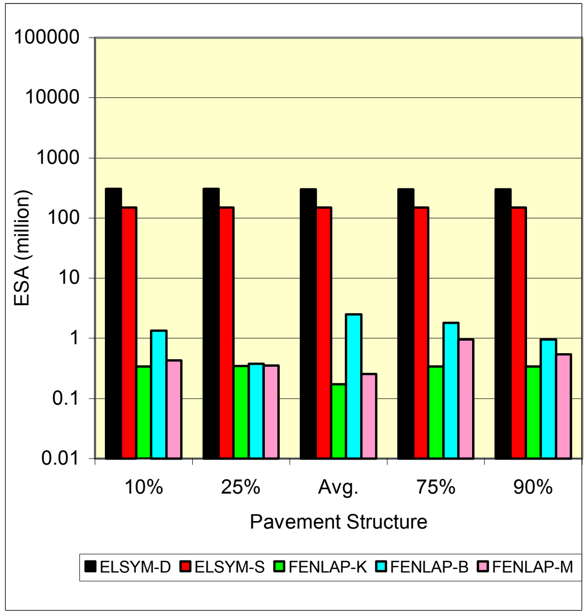
Life (ESA X10^{6}) 100 mm Asphalt @ 2100 MPa

	Variation	10%	25%	Avg.	75%	90%
Model	ELSYM-D	129	128	122	120	119
	ELSYM-S	153	152	151	150	149
	FENLAP-K	0.05	0.05	0.05	0.05	0.05
	FENLAP-B	4.09	0.32	0.03		0.17
	FENLAP-M	0.02	0.09	0.16	0.16	0.07



Life (ESA X10^{6}) 150 mm Asphalt @ 2100 MPa

	Variation	10%	25%	Avg.	75%	90%
Model	ELSYM-D	306	305	302	300	299
	ELSYM-S	150	150	149	149	149
	FENLAP-K	0.34	0.34	0.17	0.34	0.34
	FENLAP-B	1.34	0.38	2.52	1.80	0.95
	FENLAP-M	0.43	0.36	0.25	0.95	0.54



Pavement Structure No. corresponds to those pavements listed in the earlier tables

9.3.4 Comparison 5 - Variation with the Introduction of a Random Error into the Strain Measurements

For this sensitivity analysis, a random error was applied to actual strain measurements for variations ranging from small (2%) to large (50%). The analysis was conducted and the pavement life predictions made using the mechanistic design methods. It was found that little variation in the material parameters and coefficients resulted.

The results of the pavement analyse predicting design life yielded much the same general trend with respect to the models, as shown in Figure 9-12. Little change in traffic loading predictions occur between 0% and 50% increase in the magnitude of the random error. For this analysis it was noticed that, in general, the difference between predictions made by the different models are not as great as those for Comparisons 1-4. This substantiates the conclusion from the previous chapter that a random error in the strain data during the testing procedure has little or no effect on the final pavement design outcome.

9.4 SUMMARY

The mechanistic design of pavements attempts to model the interaction of various layers comprising materials in a pavement structure. The South African Mechanistic Method (SA-MDM) was chosen for use since this method considers failure in unbound granular base layers.

Numerous mechanistic pavement design analyses were conducted using a linear elastic method (ELSYM5) and a non-linear method (FENLAP). Both are computer programs that calculate the stresses and strains at various locations within a pavement structure under traffic loading.

An 80 kN axle load is simulated by the analysis, however dual wheel loads cannot be used in FENLAP, therefore a comparison of single and dual loads was conducted using ELSYM5.

Figure 9-12 Comparison 5 - Variation with the Introduction of a Random Error into the Strain Measurements

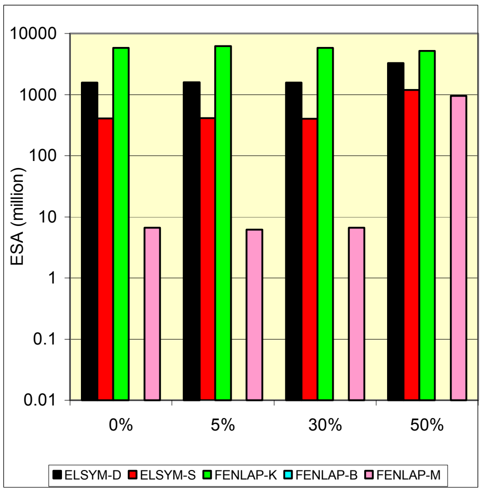
Pavement life as a function of the Range of values in the Base Characteristics when a Random Error is Introduced into the Strain Measurements

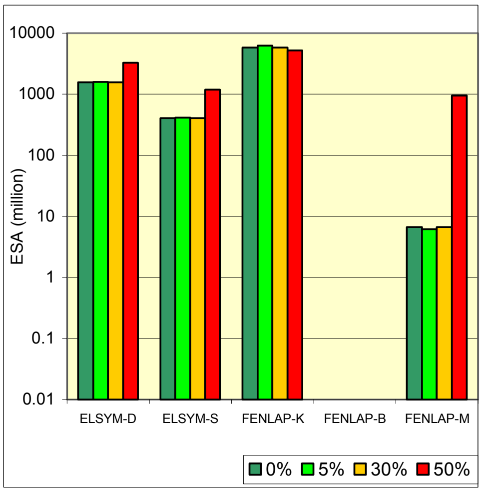
Life (ESA X10^6) 100 mm Asphalt @ 2100 MPa

	Variation	0%	5%	30%	50%
Model	ELSYM-D	1563	1586	1563	3290
	ELSYM-S	407	413	405	1194
	FENLAP-K	5809	6201	5809	5206
	FENLAP-B				
	FENLAP-M	6.65	6.15	6.65	951.59

Life (ESA X10^6) 100 mm Asphalt @ 2100 MPa

	Pavement	0%	5%	30%	50%
Model	ELSYM-D	1563.07	1585.79	1563.07	3290.25
	ELSYM-S	406.56	412.77	404.51	1193.98
	FENLAP-K	5809.38	6201.12	5809.38	5206.04
	FENLAP-B				
	FENLAP-M	6.65	6.15	6.65	951.59





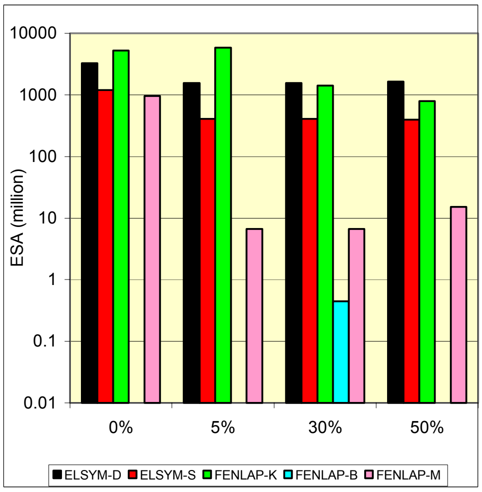
Pavement life as a function of the Range of values in the Subbase Characteristics when a Randon Error is Introduced into the Strain Measurements

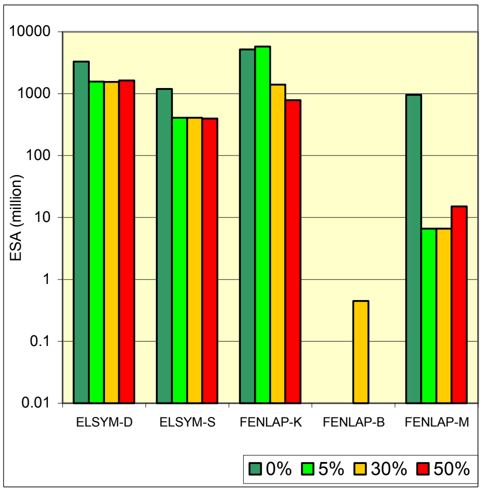
Life (ESA X10^6) 100 mm Asphalt @ 2100 MPa

	Variation	0%	5%	30%	50%
Model	ELSYM-D	3290	1557	1549	1642
	ELSYM-S	1194	408	409	394
	FENLAP-K	5206	5809	1406	786
	FENLAP-B			0.45	
	FENLAP-M	951.59	6.65	6.65	15.16

Life (ESA X10^6) 100 mm Asphalt @ 2100 MPa

	Pavement	0%	5%	30%	50%
Model	ELSYM-D	3290.25	1556.66	1549.21	1641.53
	ELSYM-S	1193.98	407.79	409.03	394.08
	FENLAP-K	5206.04	5809.38	1406.26	785.54
	FENLAP-B			0.45	
	FENLAP-M	951.59	6.65	6.65	15.16





Some typical values for resilient modulus were quoted in an earlier chapter. In general good quality crushed rock should have a resilient modulus of between 100 and 600 MPa, whereas a subgrade soil should have a resilient modulus of between 20 and 200 MPa. The results for the three test programmes generally yielded values within these ranges for the materials tested, although the soils, tested at LNEC, under Test Programme III resulted in high characteristic resilient moduli.

Five comparisons were made by varying material properties and conducting mechanistic pavement analyses to determine the pavement life in ESA. The main conclusions from these comparisons are:

- Pavement life increases with asphalt thickness for thick surfaces (100 mm and 150 mm) for the same pavement structure and material characteristics. For thin surfaces (50 mm), however, when analysed using linearly elastic models the pavement life often exceeds that of identical pavements with thicker surface layers. This can be explained by the fact that the linear elastic analyses assume that thinner surface layers are more flexible and thus able to withstand higher deflections, stresses, without significant failure.
- Vastly different predictions of life were calculated by the linear elastic models to those of the non-linear methods. This worrying revelation indicates that one of the methods is not modelling the pavement correctly. This is substantiated by the fact that predicted surface deflection bowls showed large deflection variations between linear elastic and non-linear modelling methods.
- Dual loads were found to cause more damage to thinly surfaced roads than equivalent single loads. This is due to the coincidence of stresses, thus increases in stress magnitude, at the bottom of the thin surface, which is the critical area, for dual wheel loads. This concentration of stress occurs in the centre or upper part of the thicker surface pavements, an area that is less critical.
- Variations in the base quality (198 to 245 MPa) had little effect on the predicted life, thus the results from the four laboratories in Test Programme II resulted in similar pavement design predictions. This is substantiated by the fact that when a

greater range of base material properties were compared against one another the effect on the pavement life was much greater.

- Variations in the subgrade quality (8 to 48 MPa) also had little effect on the predicted life of the pavement. However the variation of this material was much greater (6 times) and therefore it is surmised that this layer has much less influence on the design life of the pavement, at least when tensile asphalt strain is the parameter controlling pavement life. This is substantiated by the outcome of the sensitivity analysis which concluded that there was little change in design life with large variation in the material properties of the subgrade layer.
- Poisson's ratio values, where required, have a marked effect on the modelled pavement life predictions and care must be taken to estimate this parameter correctly.
- During the non-linear modelling analysis the complex Boyce and Mayhew models often fail to produce a result, particularly when the surface layer was thin (50 mm). Conversely the k-theta model proved to be very robust as were the linear elastic analyses.
- Large variations to the random errors in strain measurements at the testing level had very little effect on the ultimate predicted life of the pavement. This implies that the material models at all levels are able to cope with large variations in the basic data used to predict the material properties over a large range of stress paths.

10 SUMMARY AND CONCLUSIONS

10.1 SUMMARY

All of the laboratory work and much of the analysis reported in this thesis was conducted while the author was employed as a research assistant to work on the 'Science Project' between 1990 and 1993. During this period he was based in Lisbon, Portugal.

The author visited all of the participating laboratories and conducted substantial repeated load triaxial tests, comprising five test programmes at LNEC, UNOT and LRSB using the repeated load triaxial apparatus of various configurations, and varying instrumentation, to measure the deformation of the specimens under loading. He was particularly involved with the development of the 'String of Wheels' for the measurement of radial deformation of specimens.

This work uses the data obtained during these test programmes to identify and quantify errors involved in unbound material testing (subgrade soils and granular materials) and goes some way to identifying the consequence of these errors on the final pavement design.

A number of materials, that are indicative of road construction materials across Europe, were selected and tested and the results have been analysed. These materials comprised both subgrade soils, which are used in road foundations, as well as unbound granular materials that are used in the upper layers (subbase and base). An artificial material was also tested in order that comparisons could be made between repeated load triaxial apparatus, and the various instrumentations, independently of the material characteristics.

As a result of the detailed instrumentation comparison in this work the magnitude of potential errors are quantified and certain recommendations are presented and conclusions made.

The objective of repeated load triaxial testing is to produce material parameters that characterise the materials, such as resilient modulus and Poisson's ratio. However, it is recognised that these road construction materials are heavily stress dependent and therefore a single value of these parameters to describe a material is inadequate. Nonetheless, common analytical methods used for practical pavement design require such single material parameters. Furthermore, engineers favour assigning values to materials so that they can easily rank various material and pavement quality options. It is with this in mind that the author has chosen a particular stress level to define the quality of materials rather than using some arbitrary stress level, for example that of Paute et al (1986). He has determined a *'reasonable'* stress level as applied to the base and the subgrade of a *'reasonable'* pavement structure under *'reasonable'* traffic loading and defined this as the *'characteristic stress'*. Analytical analysis, using mathematical models and applying characteristic stress value results, were undertaken to obtain characteristic material parameters (resilient modulus, Poisson's ratio, volumetric and shear strain). A simple iterative analysis shows that the characteristic stresses are insensitive to the change in the initial assumed material parameters. Although this definition of characteristic stress remains open to some criticism, it does result in a numeric parameter with which comparisons of materials and, indeed, pavements comprising these materials can be made.

The repeated load triaxial test data has been analysed using seven previously published material models. These models attempt to describe the behaviour of road construction materials under traffic loading, and all require material coefficients that are established for the particular material. These coefficients have been established from repeated load triaxial testing.

Two different numerical analysis methods were used to determine the model coefficients for these models and various materials. The choice of the method was dependent on the complexity of the model being considered. A range of pavement structures of *'reasonable'* layer thicknesses comprising the material parameters, as characterised by the material testing programme, were analysed.

The stresses and strains obtained from the analytical methods were then applied to the South African mechanistic design method, which was selected as a suitable pavement design method, enabling the pavement life for each different pavement structure to be established. This allowed the author to make comparisons of the different analytical methods and material models.

10.2 DISCUSSION

Apparatus and Instrumentation

Of the eight repeated load triaxial apparatus contained in five laboratories, considered in this work, seven vary to some degree. Importantly, there is a high variability between the instrumentation, which measures stresses and strains.

There is no system that clearly stands out above other systems as giving improved performance. Indeed most systems have been developed because of certain needs or preferences within the particular laboratory. They all use some form of electronic transducer, or strain gauge, to measure the movements and stresses and capture the data using an electronic device.

During this work, some basic guidelines have been established with respect to repeated load triaxial testing of subgrade soils and unbound granular materials in the apparatus as follows:

a) The axial load cell should be placed on the loading rod inside the cell in order to avoid the effects of friction between the rod and the cell.

b) Variable confining pressure repeated load triaxial apparatus are desirable but require a cell surrounding the specimen. This makes accessing the instruments during a test difficult.

c) Constant confining pressure repeated load triaxial apparatus are generally used for large specimens (for testing large particle material such as granular subbase and base materials) and instead of a surrounding cell an internal vacuum is used. Unfortunately, with these apparatuses the confining pressure cannot be varied but the instrumentation can be easily accessed. Also, larger specimens require more material, are more time consuming to fabricate, are more difficult to manoeuvre and the repeated load triaxial apparatus required to test them is much larger and thus, more expensive.

Sample instrumentation may be fixed to the specimens by a number of different methods. Placing measurement studs or pins into the specimen provides a positive method of measurement of axial specimen deflection, which eliminates the possibility of slip, which could conceivably occur between a spring-loaded clamp and the membrane. The major drawback associated with using studs or pins in a granular material is that specimen preparation is greatly complicated because of the presence of a stud (or pin), which protrudes both into and out of the specimen. This is of greater significance during the preparation of a granular specimen, since studs must be affixed to the mould during specimen compaction, which can cause problems with the material density around the studs.

a) Axial deformations measured from the top platen give erroneous results due to end effects between the specimen and the platens. Measurement should be taken some distance from the end platens. Commonly this is conducted between one third and two thirds of specimen height or between quarter points. The greater gauge length obtained from quarter points does result in larger deformations, which will result in a more reliably measured strain reading. Three measuring points should be positioned at 120° to one another around the specimen, in order that any discrepancies such as tilting can be detected. However, the laboratories assessed in this work have not always been found to be practical due to extra instrumentation requirements and space within the cell.

b) Similarly, radial measurement should be taken within the third or quarter height of the specimen, to eliminate the end effects. As with axial deformation measurement, instruments should be positioned at 120° to one another but, again, this has not always been found to be practical.

c) Some axial measuring LVDTs are glued onto the membrane, which surrounds the specimen, while others are attached to studs embedded into the specimen, penetrating the membrane. These two methods of instrumentation attachment did not show great discrepancies with each other. However, care should be taken if radial measuring apparatuses are glued onto the membrane because the changing of pressure, in the cell or within the specimen, may cause membrane compression and consequent misreading of radial strain.

An experiment with an artificial specimen tested at each of the laboratories, and at one laboratory with multiple instrumentation, has given some confidence that different instrumentation systems can give similar, although not identical, results. Measurements with instrument influenced variability in the ranges of ±5 to ±10% of the mean strain value should be expected.

These artificial specimen tests could not use embedded fixings and did not use glue-on fixings. Embedded fixings cause some disturbance to the specimen and this is thought to be a further contributor to differences between instrument outputs, although this could not be assessed completely independently of other variables. For many purposes embedded fixings are preferred, as they avoid membrane interaction problems.

It is very important that some understanding of the possible errors and inaccuracies of the particular system is undertaken by monitoring and calibration. An example of this is shown by the fact that digital noise was found to account for strain measurements of up to 90με during this study.

It is therefore concluded that laboratories should conduct an assessment of their instrumentation and define the error for each instrument. It may be possible to periodically check the entire test apparatus and instruments using an artificial specimen of known mechanical properties as a reference.

It has been shown that the mean radial measurement for testing three specimens each manufactured and tested at four laboratories is -140 με, the standard deviation is 68 με and the coefficient of variation is 49%, which is a great deal poorer than the 12% value of the axial resilient strain measured between the laboratories. It is thus concluded that there are greater errors in the radial measurement systems than the axial measuring systems.

For the unbound granular material, the axial resilient strain results show little systematic difference, although improved readings appear to result from a larger specimen size. The variability in readings is particularly high for the UNOT tests (which may be due to stud rotation generating apparent strain - sometimes increasing, sometimes decreasing the measured values above the average obtained at all the laboratories). For radial resilient strains all laboratories yielded a large scatter in strain values. There was no systematic variation in radial strains between laboratories.

Tests conducted on the artificial specimen shows that there is a substantial variation in the loads (stresses) applied to the specimens and this will have an obvious effect on the strains. It is, therefore, necessary to take actual stress values into consideration when comparing and analysing results rather than to simply assume that the stress levels specified were achieved.

Despite the advice offered here, it is clear that the ‘best’ performance will still contain many uncertainties and inexactitudes that are due to a whole range of factors. The value of inter-laboratory comparisons of the type recorded here is high. Systematic errors were highlighted, procedures crosschecked and quality generally improved.

Compaction Methods

It was found during this study that large differences in the test results occurred when identical materials were tested at different laboratories.

a) These different results were shown to be due to differences in the compaction method used by the laboratories, rather than the apparatus and instrumentation used to capture the data.

b) Methods of compaction which induce high levels of shear, such as the vibrating hammer (LNEC) and manual tamping under cyclic preloading (DUT) result in lower permanent strains than the methods where the compaction is full face, inducing less shear, such as the ‘vibrocompression’ apparatus (LRSB) and vibrating table and full face static load (UNOT). Thus, the method of compaction is an important factor in attaining consistent results between repeated load triaxial tests. Therefore, it is imperative that a standardised method of specimen manufacture and compaction is established between laboratories if the results obtained from testing are to be compared and applied to a single pavement design method.

Removal of *'Poor'* Data

The author has devised and described a method for removing outliers from the test data based on the difference between the modelled and experimental material parameters for each stress path applied to a particular specimen. After considering a number of degrees of data exclusion it is concluded that, in general, for the test procedures used (stress paths applied), better correlation between the modelled and experimental data is obtained when the 'worst' 10% of the data is removed.

Critical Locations in a Pavement Structure Associated with Failure

Most pavement design procedures are primarily concerned with the two critical strains values namely the:

a) Horizontal tensile strain at the bottom of the asphalt layer (to limit asphalt fatigue cracking), and;

b) Vertical compressive strain at the top of the subgrade (to prevent excessive permanent deformation).

Of the methods reviewed herein, only the South African design method presented a means of assessing the shear deformation and failure in the unbound granular layers for subbases and bases in pavements. However, the shear deformation and failure in the unbound granular layers for subbases and bases in pavements was not found to be critical for the pavement structures considered herein and it is therefore concluded that the two criteria, above, are satisfactory.

Introduction of a Random Error to Test Data

The introduction of a random error at different variations (up to 50% of the measurement) was found not to affect the final outcome of the analyses when the variation was below 30% but to rapidly increase once 30% was exceeded. This has an important consequence for the accuracy at instrumentation level. It implies that some variations found in measurements using electronic instrumentation, such as random electronic noise, may not be as important as the inaccuracies generated by other aspects of the testing, such as specimen manufacture.

Comparison of the Test Results from the Test Programmes

This work investigated the effect that using two different methods of analysis have on the resultant characteristic material parameters using a single set of test results. SFB, which for this work is defined as dry single sized sand, is considered to behave as a subgrade soil as well as an unbound granular material under loading. Therefore, the two methods of analysis, used in this work, were applied i.e. the behaviour can be modelled as subgrade soil and as an unbound granular material. This means that all seven of the material models considered in this work could realistically be applied to the same set of SFB test results obtained from each laboratory.

a) It was concluded that there is greater variation between the resilient moduli as predicted from analysing the results from a single specimen (from one laboratory) using different models than there is when the results from different specimens are analysed using a single model. Therefore it seems more important to select an appropriate predictive model and analytical method than is it to obtain test results which are 'close' to each other.

b) An investigation was conducted regarding the effect that the variation of the testing of a granular base material at four different laboratories made on the pavements' life. It was found that very little variation in the pavements life results from using the material parameters from the various laboratories. It was noted that the actual results from the laboratories were close to one another.

c) Similarly, a study of the effect on the pavement's life with the variation of testing a fine-grained subgrade soil at four different laboratories concluded that although the test results from the laboratories were quite varied, the different material parameters made an insignificant amount of difference to the predicted pavement life.

d) An investigation was also conducted regarding the effect on the pavement's life due to the variation of the results when testing a granular material (base or subbase) at a single laboratory. It was found that a considerable difference was found in the predicted life of the pavement when the more simple linear elastic analysis is conducted using the material parameters obtained, whereas very little variation in pavement life was found for the more complex non-linear method. It must be noted that the actual variation between the results from tests of different specimens at a single laboratory was found to be considerable.

e) Similarly, a study was undertaken as to the effect on pavement life caused by the variation of the results obtained from testing the fine-grained subgrade soil at a single laboratory. It was found that very little difference was found in pavement life even though the variation in the results obtained is considerable.

Based on the above five points it is concluded that it is not as important to conduct detailed, and expensive, testing and analyses on materials that are to be used in the lower foundation layers of a pavement (subgrade soils) as it is on the upper granular layers (subbase and base).

These studies show that not only are there significant differences between the results obtained from specimens tested at different laboratories but also between specimens tested at a single laboratory. Most of the laboratories were not conducting regular test programmes using these apparatus, and the fact that procedures were not as refined or efficient as they might be may go someway to explaining this. With the introduction of standard specifications for repeated load triaxial testing of road construction materials efficiency and repeatability should improve with time and familiarity.

<u>Comparison of Pavement Structure Incorporating the Material Parameters</u>

Five comparisons were made by varying material properties of different pavement structures and conducting a mechanistic pavement analysis to determine the pavement life (ESA) based on a common method. The main conclusions from these comparisons are:

a) Pavement life increases with asphalt thickness for thick surfaces (100 mm and 150 mm) for the same pavement structure and material characteristics. For thin surfaces (50 mm), however, when analysed using linearly elastic models, the pavement life often exceeds that of identical pavements with thicker surface layers. This can be explained by the fact that linear elastic analyses, when horizontal asphalt tensile strain is critical, compute small strains in thinner surface layers due to their greater flexibility. Thus the layer is able to withstand a greater number of load applications before failing.

b) Dual wheel loads were found to cause more damage to thinly surfaced roads than equivalent single loads. This is due to the coincidence of stresses at the bottom of the thin surface, which is the critical area, for dual wheel loads. This concentration of stress occurs in the centre or upper part of the thicker surface pavements, an area that is often considered less critical than the bottom of these layers, for example.

c) The linear elastic layered analyses predict very different pavement lives to those of the non-linear finite element methods. The reason for this has not been fully determined but seems to indicate that at least one of the methods is not modelling the pavement correctly. This is substantiated by the fact that predicted surface deflection bowls showed large deflection variations between linear elastic and non-linear modelling methods. It is known that ELSYM5 does not correctly calculate stresses and strains for unbound materials (i.e. it allows tension). This method, however, has benefited from long term use and substantial field verification and would appear to predict more reasonable pavement lives. The FENLAP analytical method would benefit from full-scale validation in order to establish whether the low predictions are realistic and if not, what factors should be included in the computations which are currently ignored.

d) Variations in the base quality (characteristic resilient modulus between 198 and 245 MPa) had little effect on the predicted life, thus the results from the four laboratories in Test Programme II resulted in similar pavement design predictions. This is substantiated by the fact that when a greater range of base material properties were compared against one another (in Programme I) the effect on the pavement life was much greater.

e) Variations in the subgrade quality (characteristic resilient modulus between 8 and 48 MPa) also had little effect on the predicted life of the pavement. However the variation of this material was much greater (6 times) and therefore it is concluded that this layer has much less influence on the design life of the pavement. This is substantiated by the outcome of the sensitivity analysis which concluded that there was little change in design life with large variation in the material properties of the subgrade layer.

f) Poisson's ratio values, where required, have a marked effect on the modelled pavement life predictions and care must be taken to estimate this parameter correctly.

g) During the non-linear modelling analysis the complex Boyce and Mayhew models often fail to produce a result, particularly when the surface layer was thin (50 mm). Conversely the k-theta model proved to be very analytically robust as were the linear elastic analyses.

h) Large variations (random errors) in strain measurements at the testing level had very little effect on the ultimate predicted life of the pavement. This implies that the material models at all levels are able to cope with large variations in the basic data used to predict the material properties over a large range of stress paths.

10.3 CONCLUSIONS

The overall conclusions that were obtained from the triaxial testing of road construction materials and the subsequent analysis of the results as described in this thesis are that:

a) *Although the resilient modulus characteristics for the subgrade soils from the different laboratories varied by a factor of 6, applying these different material parameters to pavement designs made an insignificant amount of difference to the predicted pavement lives. It is therefore concluded that sophisticated and expensive repeated load triaxial testing of materials in the lower layers (subgrade soils) is not beneficial as far as pavement analysis is concerned, because, for most pavements used here the wide soil variability has little effect on pavement life. Less complex laboratory tests might, therefore, be employed for testing subgrade soils and, similarly, the analysis of their results to produce the material characterisation can be simple without loss of relevant pavement design precision.*

b) *The range of the experimental resilient modulus values for the soils was found to be somewhat less than the range of the values estimated by the models for these materials. However the range of the experimental values for the unbound granular materials was found to be approximately equal to the modelled values. This implies that it is not as important to use sophisticated models for lower layers (subgrade soils) as it is for the upper layers in a pavement, where unbound granular materials are commonly used. Hence, this supplies a second reason for giving more attention to the characterisation of the material in the upper layers (UGM) than to the characterisation of lower layers (soils).*

c) *Random instrumentation errors in the range ±30% are less concerning than small bias errors as the implicit averaging process which occurs when fitting a material model to the collected readings, minimises their impact on the ultimate computed pavement life. Therefore during the examination of test* *data more effort should be given to identifying, and removing, systematic (bias) errors rather than scatter (noise) errors.*

d) *There is a major effect on predicted pavement life depending on the particular selected analytical method. It is suggested that only analytical techniques for which performance criteria have been developed and validated, preferable against full scale trials, are useful.*

e) *Generally, simpler constitutive relationships give acceptable fits to laboratory data. Given the difficulties in applying complex models and the uncertainties and errors experienced elsewhere it is recommended that these simpler models are normally used.*

f) *It has been observed that the computed life, and hence design thickness, of a pavement is much more sensitive to the material model used to describe the behaviour of the laboratory soil or aggregate specimen under loading and to the analytical pavement method (ELSYM or FENLAP) than it is to any variations in material behaviour likely to be observed with or between laboratories. In the light of the previous conclusions it is, therefore, concluded that the greatest care should be taken to select the most appropriate analytical procedure.*

In addition to these, useful conclusions obtained from this work are:

- *Variability of strain readings in the range ±5 to ±10% of the mean strain value should be expected from on-sample instrumentation.*
- *Greater error magnitudes (approximately 4 times) occur with the radial measurement systems than with the axial measuring systems.*
- *Different results from different laboratories were shown to be due, largely, to differences in the compaction method used by the laboratories rather than the apparatus and instrumentation used to test the specimen and capture the data.*

- *Better correlation was found between the modelled and experimental data when the 'worst' 10% of the data is removed. Therefore the removal of 10% of 'outliers' using the method described herein, is recommended in future.*
- *Poisson's ratio values, where required, have a marked effect on the modelled pavement life predictions and care must be taken to estimate or measure this parameter correctly.*
- *Pavement life predictions are much more sensitive to variations in soil and aggregate characterisation when using a linear elastic layered analysis (ELSYM) than when using a finite element method (FENLAP). Is it noted that the older linear elastic approach has benefited from field validation and therefore given (d), above, calibrating the pavement life predictions of the newer finite element methods directly to observed performance should improve the practical application and usefulness of these methods.*
- *Test repeatability was found to be poor with a coefficient of variation about the mean for soils having a typical range of ±80% and up to ±170% for UGM. Together with conclusions (a) and (b), above, better predicted pavement lives/ thicknesses would be obtained if more accurate test results were obtained during testing of UGM. However, the coefficient of variation about the mean for models is considerably better with a typical value of ±20% for soils and ±40% for UGM. This implies that the predicted designs for soils based on existing tests methods and modelling methods are satisfactory but that, better predicted answers would be obtained if more accurate test results were obtained during testing of UGM although the models mask this inaccuracy somewhat.*

10.4 RECOMMENDATIONS FOR FUTURE WORK

Specimen Manufacture

A detailed investigation should be conducted on the various methods of specimen manufacture and, importantly, compaction. Based on the conclusions above it is apparent that a uniform method of specimen manufacture is required. This work should aim to draw up a detailed testing specification with the compaction and specimen manufacture for various road construction materials. It is recommended that a compaction standard be formulated that does not just consider the ease of 'producing' specimens in the laboratory but also a procedure that manufactures specimens that closely replicate the in-situ conditions.

Instrumentation Advances

Since the laboratory testing was conducted for this work (1990 to 1993) less expensive, commercially available, instrumentation with much improved accuracy is certainly available. This is particularly desirable for radial measuring instrumentation which was shown to be less accurate than the instrumentation methods that measure axial strain. This instrumentation will improve the ease with which these sophisticated laboratory tests might be conducted. Advances in this instrumentation, as well as the entire apparatus within which the specimen is tested and the method by which they are employed, should be continually monitored.

Characteristic Values

It is recommended that further work be undertaken in deriving acceptable 'characteristic' values. These should not just be the stress values for materials based on the expected depth in the pavement of these materials but also the predicted acceptable material parameters, for example resilient modulus and Poisson's ratio. If acceptable models were defined for material this could be extended to defining acceptable characteristic model coefficient values as well.

Analytical Modelling

Numerous analyses were conducted for a range of road construction materials resulting in the material properties (material parameters and model coefficients) being produced for certain analytical models. During this work some problems in determining realistic solutions to some models was encountered and a pragmatic set of rules (minimum and maximum values for the coefficients and parameters) was formulated. This allowed for the characterisation of 'good' and 'bad' results, where the bad results were unrealistic and could be removed. Future work might be to refine these rules with respect to the behaviour of road construction materials.

The mathematical models used in this work fall short of perfectly predicting the behaviour of the materials under repeated loading. Although perfectly correct modelling may never be achieved, largely due to the fact that repeated load triaxial testing does not correctly simulate the true pavement situation, with the increase in capabilities of computing methods models should be continually improved.

Analytical Pavement Design Methods

Substantial differences in the predicted life for the same pavement structure resulted from the two analytical methods used to determine the stresses and strains at certain critical points in the pavement structures. ELYSM5 predicts that the structure will carry substantially more traffic than the predictions made by FENLAP. It is known that ELSYM5 does not correctly calculate stresses and strains for unbound materials (i.e. it allows tension). This method, however, has benefited from long term use and substantial field verification. The FENLAP analytical method would benefit from full-scale validation in order to establish whether the low predictions are realistic. If a reliable source of field data could be obtained (for example that of the HVS in South Africa) then these methods (and others that are available) and their respective predictions regarding the actual field occurrences could be compared to one another.

Standard Specifications

The Science Project has resulted in the formulation of a standard specification for repeated load triaxial testing of subgrade soils and unbound granular materials which has formed the basis of a new CEN standard to be implemented across Europe CEN (2000). Similarly a standard specification is being applied in the USA, Australia and probably other countries. Future work might investigate the success or otherwise of the implementations of these specifications worldwide and make recommendations for improvement based on the past decade's experience. Importantly, this should be applied from the specimen manufacture stage through to the pavement design phase.

The use of an artificial specimen resulted in some important findings in this work. This could be extended to making firm recommendations as to the composition of such a material and its use for calibration of repeated load triaxial apparatus worldwide. A database of the results from instrumentation and apparatus calibration would benefit all users.

Acceptable Errors

During this work the errors that occur during repeated load triaxial testing and the subsequent analysis of the test results resulted in some revelations, for example, the introduction of a random error at different variations had little affect on the final outcome of the analyses. It is recommended that all future work consider the potential errors in testing and analysis and then clearly define such errors. The production of standard specifications should clearly state what errors magnitudes are acceptable and what action to take if unacceptable errors are found to occur. Implementation of such an approach is likely to lead to changes in specimen preparation, test procedure and data processing. Each stage needs to be assessed against the error variations which stem from it.

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12 APPENDICES

The appendices are contained on a Compact Disk in Adobe Acrobat (pdf) format bound into the back of this volume together with a copy of Acrobat Reader Version 4.

This document (i.e. the thesis) is also contained on this Compact Disk in Adobe Acrobat (pdf) format.

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Appendix H Mechanistic Pavement Design

Appendix H.1 Mechanistic Pavement Design Analyses

Appendix H.2 Pavement - Life Estimations